NUMERICAL MODELING OF BED CHANGES IN ALLUVIAL CHANNELS CONSIDERING NONUNIFORM BEDLOAD SEDIMENT

BY

Khalid Abdel Fattah Mohamed Osman

Thesis submitted for the requirement of the Degree of Doctor of Philosophy in Civil Engineering

To

Department of Civil Engineering

Faculty of Engineering & Architecture

University of Khartoum

January 2005

DEDICATION

To my Family

ACKNOWLEDGMENTS

I would like to express my grateful acknowledgment and deep gratitude to Dr. Elsadig Mohamed Abdalla Sharfi, the advisor of this research, for his valuable ideas and helpful suggestions with regard to search in this field. His unlimited encouragement during the various stages of the progress to execute the work is sincerely appreciated.

My thanks extended to Dr. Mohamed Akode Osman for his valuable comments to extend some areas of the research. Gratitude is expressed to Professor Siddig Eissa for his comments and suggestions, and to the staff and postgraduate students of Water Resources Engineering Division, Department of Civil Engineering, University of Khartoum, for their support and discussions.

Special thanks expressed to Eng. Hisham Isam for his continuous support and collaboration.

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LIST OF SYMBOLES

u_i $(i = 1,2,3)$ = Velocity components [m/s]

A = Flow area [m^2]

τ_{ij} = Turbulent stresses [N/m^2]

g **= Acceleration gravity [m/s^2]**

$\bar{h}$ = Average flow depth [m]

h_c = Critical flow depth [m]

τ_c **= Critical shear stress [N/m^2]**

τ_c^* = Dimensionless critical shear stress parameter

q = Discharge per unit width [m^3/s/m]

x = Distance step [m]

F_i = External force per unit volume of fluid [N/m^3]

h_β **= Modified critical flow depth [m]**

h_n = Normal flow depth [m]

η = Sediment characteristics parameter

S_t = Sediment transporting energy slope

ν **= Kinematic Viscosity [m^2/s]**

θ, ϑ = Weighting coefficients

v = Flow velocity [m/s]

h = Water depth [m]

A_b = Area of the bed [m^2]

z = Bed level [m]

δ_s = Bed material layer of thickness [m]

E_m = Bed material thickness [m]

S_o = Bed slope

B = Channel width [m]

k_a = Coefficient of active pressure

β = Critical depth modification factor

Q = Discharge [m^3/s]

h_a = Energy head loss due to acceleration [m]

ρ = Fluid density [kg/m^3]

P = Fluid pressure [N/m^2]

τ'' = Form shear stress [N/m^2]

h_f = Friction energy head [m]

S_f = Friction energy slope

Fr = Froude number

R = Hydraulic radius [m]

K_l = Loading-law coefficient

n = Manning coefficient [m$^{-1/3}$s]

h_α = Modified normal flow depth [m]

α = Normal depth modification factor

p = Porosity

d_s = Representative diameter of sediment particles [m]

h_r = Resisting energy head [m]

R_e = Reynolds number

φ = Sediment characteristic parameter

ϕ = Sediment coefficient

R_e^* = Sediment particles Reynolds number

q_s = Sediment transport rate per unit width [m^3/s/m]

q_s^* = Equilibrium value of q_s [m^3/s/m]

τ = Shear stress [N/m^2]

τ' = Skin shear stress [N/m^2]

E = Specific Energy [m]

ss = Specific Gravity of sediment particles

γ_s = Specific weight of sediment [N/m^3]

γ = Specific weight of water [N/m^3]

E_m = Thickness of the active layer [m]

t = Time step [s]

z_a = Total aggradation [m^2]

z_d = Total degradation [m^2]

h_t = Transporting energy head [m]

λ = Water slope modification factor

y = Water surface elevation [m]

U = Flow velocity [m/s]

U^* = Shear velocity [m/s]

ω_s = Settling velocity [m/s]

ϕ_{bk} = Non-dimensional bed load transport capacity

ϕ_{skk} = Non-dimensional suspended load transport capacity

$\tan\phi_s$ = Angle of repose of sediment particles

C_s = Volumetric concentration of the sediment in the bed-load layer

$\cos\theta$ = Longitudinal bed slope

σ_e = Effective stress [N/m^2]

m_s = Mass of sediment flow [kg/m^3]

ρ_s = Density of sediment particles [kg/m^3]

ملخص

في هذا البحث، تدرس الافتراضات والاعتبارات التي توضع عادة في عملية النمذجة لمحكاة تغيرات القاع في القنوات الغرينية، ذات القاع المتحرك، نتيجةً لإنتقال طمي حمل القاع. يتم اتباع طريقة نظرية وأخرى تصورية بحثاً عن مفاهيم جديدة يمكن من خلالها تعديل المعادلات التي تحكم الظاهرة. في التطوير النظري تدرس إمكانية تعديل صيغة المعادلة للجريان المتغير تدريجياً، والتي تستخدم عادةً للقنوات ثابتة القاع، بحيث يمكن إستخدامها للقنوات ذات القاع المتحرك. وذلك بإقتراح مجموعة من المعاملات لتصحيح قيم العمق الطبيعي للجريان وعمق الجريان الحرج وميل سطح الماء. هذه المعاملات أفترض أنها ترتبط بعلاقة أسية مع معلم مميزات الطمي. مفهوم جديد يتضح من خلال البحث التصوري، هو مفهوم سمت طاقة نقل الطمي، وهو دالة في معامل يتعلق بخصائص الطمي. وفي البحث التصوري أيضاً، يتم تعديل معادلة الجريان المتغير تدريجياً للقنوات متحركة القاع من المبادى الأساسية مع الأخذ في الإعتبارمفهوم سمت النقل وخصائص الطمي المنتقل. المعادلة المشتقة،

نظرياً وتصورياً، تختبر ويتم التأكد من صحتها بإستخدام بيانات لتجارب SAFL المعملية كبيرة الحجم.

المفهوم الجديد المستنبط لميل طاقة نقل الطمي يستخدم لإقتراح صيغة جديدة لحساب انتقال طمي حمل القاع، بدلالة معلم لا بعدي لإجهاد القص الحرج في القاع. وتتم في هذا البحث صياغة نموذج SEDTREN للقاع المتحرك بإدخال المفاهيم التي تم اشتقاقها مع تطوير جديد للمعادلات التي تحكم النموذج. معايرة النموذج المقترح تتم بإستخدام بيانات حلقية لمقطع من نهر الراين في فرنسا وبالإستفادة من النتائج المعطاة بواسطة نموذج CARICHAR السابق صياغته بواسطة Rahuel et al (1989). فتتم دراسة تقييم معالم نموذج SEDTREN والتحقق من صحته. أخيراً، يطبق النموذج المقترح على حالة لمقطع من نهر عطبرة في السودان.

ABSTRACT

In this research, the basic assumptions and considerations usually proposed in modeling the bed evolution in alluvial channels due to bed load transport are investigated. Theoretical and conceptual approaches are developed searching for new concepts to modify the equations governing the phenomenon. Theoretical development of a modified gradually varied flow formula for alluvial channel is investigated through proposing a set of modification factors to correct the normal flow depth, the critical flow depth and the water slope. These factors are related exponentially to a sediment characteristics parameter. Furthermore, a new formula, sediment transporting energy head, is derived as a function of the sediment

properties and incorporated in the development of a modified flow equation. The equation is conceptually derived flow equation, developed for alluvial channels from the basic principles. The modified equation, theoretically and conceptually, is tested and verified using SAFL large-scale laboratory experiments.

The newly developed transporting energy slope is used to propose a new bed-load formula as a function of a dimensionless critical shear stress parameter. Accordingly in this research, SEDTREN mobile-bed model is formulated in which the new developments are considered. The model is calibrated using field data of the Rhone River reach, in France, and utilizing the results presented by a previously CARICHAR model presented by Rahuel et al (1989). Parametric evaluation of the SEDTREN model is carried and the verification is shown to be satisfied. Finally, the proposed model is applied to a simplified case for Atbara River reach, in Sudan.

CHAPTER ONE

INTRODUCTION

1-1 BACKGRAOUND

Significant amounts of sediments accumulate in hydraulic structures systems such as reservoirs, harbors, irrigation canals, etc. As a consequence, the performance of these structures will be affected by such accumulation and the efficiency will be reduced. To sustain an efficient storage capacity of reservoirs or irrigation systems has become a major aspect of importance, since new construction of such hydraulic structures is a rather complex requirement due to environmental regulations restrict, high construction expenses and the lack of suitable construction sites.

Obviously, Natural conditions and man-made activities affect the alluvial channels; as a result the bed level will be under a state of continuous evolution. In most cases, the man activities destroy the natural equilibrium of such channels and produce the morphological changes. Generally, the natural and artificial activities such as channel meandering, bank erosion, bed material mining, water storage in reservoir, water diversion for hydropower, etc represent the driving force of the channel bed evolution. This phenomenon is a consequence of the interaction between the sediment particles and the flowing water.

The transport of the granular material, such as silt, sand and gravel, by the channel flow determines the evolution of the channel bed. Consequently, this exerts a considerable influence on the formation of the topography of the bed and its material composition. The altering factors always attempt to achieve a new equilibrium situation between the hydrologic conditions and the sediment transported in the channel. Nevertheless, the need for predictive methods to determine the responses of channels to artificial activities is considered as important article in the mechanics of sediment transport.

Sediment waves are produced on the channel bed as a result of the sediment load movement. These are of interest in many rivers because of the threat they pose to human development, flood protection, reduction of reservoir capacity, etc. Long-term monitoring of cross-section and longitudinal profiles of the channel can document the movement of such waves. Using field data became the basis for investigating such research on the sediment waves formation, their rate of movement and the morphological changes associated with wave propagation and migration.

Sediment yields usually indicates the effects of land changes on sediment production. However, the sediment yields measured at a stream channel may not reflect the increased erosion rates upstream. Significant increase in sediment stored temporarily in a river channel or its flood plains can lead to complex changes in channel bed formation, which can subsequently change the patterns of sediment transport. The frequency of sediment movement and the distance moved depends on several factors such as the particle size of the sediment location of the sediment within the channel, the hydraulic conditions, etc.

Land changes in watersheds frequently cause changes in the size distribution of the channel bed substrate. In a part of the channel, the coarser size layer controls the initiation of the sediment transport and particles mobility. The spatial variability of the channel bed represented in the sorting of bed particles during recessional flows can affect size-selectivity in bed load transport during subsequent rising stages. In addition, many studies are addressed to the role of non-uniformity and unsteadiness of flow in alluvial channels and its influence on sediment particles sorting and deposition.

In general, transport of sediments in rivers and channels involves theoretical approaches based on simplified and idealized assumptions. Empirical methods emphasize only certain number of parameters, which considered by their authors, are to be more relevant. This implies that, the application of certain formula under field conditions is not only based on the theoretical formulation, but also on the data used in its development and calibration. On the other hand, sediment transport data available in the literature are mainly limited to laboratory experiments with only small amount of reliable field data can be obtained.

The sediment transportation regime as a term of a wide base includes the initial condition of movement, the development of sediment waves on the channel bed, the boundary conditions of their formation and the variation in bed load transport. These sediment regimes are considered to be of a complex nature as the variables involved are interrelated. The difficulty of the sediment transport problems is not only because of the complexity of the problem, but also because of the little amount of the observation data available. Thus, the application of the computational techniques developed in last century was the essential procedures to simulate such problems.

Numerical modeling of the flow in rivers is the continuously accepted engineering tool, whose evaluation can be compared to that of physical modeling. Simulation of flow conditions by numerical modeling is based on the formulation and solution of the mathematical relationships expressing the hydraulic principles governing the phenomena. Long-term riverbed evolution phenomenon due to sediment transport became more and more an important part in river flow modeling. Sediment transport modeling technique, to simulate such changes, has been widely developed. Moreover, this technique brings the need for new concepts and developments on this interesting subject.

1-2 PROBLEM STATEMENT

This research is actually started with the question that is there a need to formulate a mathematical model to simulate the bed evolution in alluvial channels, although there is a large number of numerical models that have been developed and applied to several natural rivers and streams, and experimental flume channels? Part of the answer is; there is a need to investigate the assumptions upon which these models are based and their limitations. Consequently, the physical aspects considered in the phenomena and the equations mathematically governing it, could be searched starting from the basic principles.

Most of the existing numerical mobile-bed models are based on formulations expressing the interrelated sediment transport and water flow phenomena in unsteady situations. The simplest acceptable mathematical description and the mainly used systems involve a set of differential equations. These equations include continuity and dynamic equations for water flow and the continuity equation for sediment flow. Two relations for sediment transport rate and flow resistance complement the system of that physical process. In addition, it is usually considered that for an alluvial system, the hydraulic relations allow for the existence of bed form resistance beside the friction resistance.

This system of equations and complementary relations is a representative of all basic physical phenomena and has all the mathematical properties. It is important to indicate the significance of certain terms and parameters in the system. The steady state energy line slope is considered as an explicit function of the flow and channel bed characteristics when considering only friction resistance, and as an implicit function of other variables in the case of bed form resistance. Another important parameter is the non-uniformity of sediment particle size. Thus, introduction of channel bed composition is an essential parameter to simulate the bed evolution phenomena.

It worth mentioning that, the equations related to water flow are mainly derived for fixed bed channels. Thus, the steady state energy line slope used in an alluvial system is an approximate function. On the other hand, the assumption that part of the energy is consumed in transporting the sediment mentioned by many authors, however, there is no acceptable formula for bed form resistance agreed upon. This leads to investigate the part of that energy assumed to transport the sediment particles on the channel bed. To make such investigation, a conceptual sediment layer transported on the channel bed is considered to determine the part of the energy consumed in transporting the sediment layer based on basic principles.

The gradation of sediment particles in a non-uniform mixture is usually described by the size distribution. This distribution affects the bed roughness as well as the sediment transport rate, since it is directly related to the effective shear stress. Another important effect of sediment size distribution is on the bed active layer. For proper channel bed simulation, a concise knowledge and understanding of the mechanics and schematization of the solid material composition is required. Thus, in simulating the channel bed evolution, the exchange of sediment between the underlying sediment and the active layer and the formation of the stable bed layer are considered as significant parameters.

When the formerly mentioned parameters are broadly investigated on conceptual and theoretical basis, the formulation of the governing equations system has to be modified. The development of new terms or parameters should be accounted for in the modified system. Proper algorithm for the solution of the system is also required to ensure that the formulated model simulates the phenomena correctly.

1-3 THE OBJECTIVES OF THE RESEARCH

The objectives of the considered research are mainly presented as a general objective that can be achieved through several specific objectives.

1-3-1 GENERAL OBJECTIVE

The general objective of this research is to develop a numerical mobile-bed model, starting from the basic principles, in order to compute the spatial and temporal variations of the bed evolution in alluvial channels considering non-uniform bed load sediment mixture. The corresponding water level along the channel reach is to be predicted through the model. The model is to be calibrated and verified utilizing the available sediment transport data from an experimental research work and the colleted data for a natural alluvial channel system.

1-3-2 SPECIFIC OBJECTIVES

The general objective is to be executed through a series of specific objectives. Each of these objectives is devoted to be concerned with specified part of the problem in order to formulate the required model. The specific objectives are as follow:

1- To develop an expression for the energy consumed in transporting the sediment in an alluvial system.
2- To propose a bed-load predictor to modify the system of the governing equations using the slope of transporting energy line.
3- To modify the related parameters in the governing equations according to the transporting energy concept newly developed.
4- To schematize the active bed layer accounting for graded sediment exchange and to incorporate its effect in the model.

1-4 SCOPE OF THE RESEARCH

The former sections of this chapter introduced the importance of the sediment transport and mobile-bed modeling as a subject, which has a great influence in hydraulic engineering. The research problem been conducted by clearly specifying the objectives of the work. A thorough literature review is surveyed in the next chapter of this thesis. A general review of the previous work on sediment transport, in its various divisions, is elaborated. Flow resistance and the roughness in alluvial channel and their wide empirical and theoretical relations are conducted. Mobile bed hydraulics as the topic of interest is presented and the significant relations and theories are broadly introduced. The earlier work on sediment transport computation and recently developed theories by the various researchers are mentioned. The literature review includes the work conducted on the morphological simulation and the numerical modeling techniques.

In particular, chapter two of this study concerned with several parts of the subject. Part of the literature represents the descriptive view in open channel flow and the related morphological processes. Aggradation and degradation processes in alluvial channels are presented as well as the armoring process. In addition, the flow resistance and channel roughness characteristics are presented. The bed form in alluvial channels and the flow regimes are thoroughly described. The hydraulics of alluvial channels is dealt with in some details. Flow over rigid boundary channels is overviewed as most of theories applied in alluvial channels developed essentially for rigid channels. The computation of flow surface profiles and the various methods of energy slope computation are presented. The differential equations governing the open channel flow are mentioned stating the assumptions considered.

Part of the chapter concentrates on movable boundary channels. Properties of sediments and classification and types of movement are presented. The initiation of sediment motion is thoroughly described as well as the transport relations. The uniform and non-uniform sediment mixtures as a significant part of the research are clearly distinguished. The mixed and armor layers and their influence in sediment transport are also mentioned. Last part of the literature is concerned with numerical modeling as a tool of simulation. The concept of modeling and types of numerical mobile-bed models are provided in a wide scope. The governing equations of the modeling system are collectively reported as they represent the deriving tool of the system. Hydraulic and topographic discretization as basis of the formulation is described. The various computational techniques used in solving the modeling system are stated briefly. Detailed explicit and implicit types of finite difference schemes are broadly presented are clearly distinguished. Moreover, algorithm of simulation systems is mentioned.

Chapter three, concerning the approach and the methodology of the research, concentrates on the theoretical developments and the conceptual investigations work in addition to the development of the numerical model. Firstly, flow profile in alluvial channels is considered. The basic assumptions, to modify the equation of gradually varied flow in order to compute the flow surface profile in alluvial channels, are elaborated. Theoretically proposed equation is considered with the assumed modification factors. Conceptual developments are continued in this chapter representing the applications of the energy and momentum principles are extended from rigid bed channels to alluvial channels. A formula for the energy head exerted by external forces acting on water body is developed elaborating the resisting energy term. A newly developed concept for transporting energy head is derived from the basic principles. Subsequently the gradually varied flow in rigid bed channels is modified and presented. The concept of the transporting energy is incorporated in the energy equation and the theoretical development of the modified alluvial flow equation is conceptually verified and clearly elaborated.

The formulation of the numerical model as a general objective of the research is described. A general description of the model is followed by the numerical method utilized to compute the water surface profile. The bed shear stress and critical shear are considered as the presence of the transporting energy concept. Accordingly, the computation of the sediment discharge, a bed-load predictor, is highlighted. The application of the sediment continuity equation to calculate the erosion and deposition along the channel using sediment size fractions is presented. Treatment of the active layer and bed material exchange together with the sediment distribution are also described. The model contains the computation of the flow variables that are clearly presented in a flow chart algorithm to clarify the sequence of the model computations through the main program and utilized subroutines.

Chapter four of the thesis presents application of theoretical alluvial equation to SAFL experimental data accompanied by the numerical experiments. Modification factors are studied to test the proposed flow equation of alluvial channel. The numerical experiment is used in order to investigate the concept sediment characteristic parameter and to test the validity of the conceptually derived alluvial flow equation. Application of the model and the results obtained using data of an alluvial channel that has been utilized also by other researchers are clearly described and shown. A brief description of the channel considered for the model calibration is given.

Chapter five presents the application of the developed numerical model. The various cases of tests used to calibrate the model and the numerical experiments conducted in the simulation, in addition to the parametric evaluation of the model are elaborated clearly. The results of the simulation are summarized in the chapter containing the spatial and temporal variation of the bed level and the water surface profile, computation of the sediment transport rate and graphical presentation of the longitudinal profile of the channel reach including the transporting energy line.

Chapter six concludes the overall work executed. It also presents the recommendations for the further research work that may be conducted later in order to investigate other formulations. Appendices are attached at the end of the thesis to represent in more details some parts of the work.

CHAPTER TWO

LITERATURE REVIEW

2-1 MORPHOLOGICAL PROCESSES IN OPEN CHANNELS

Transport of granular material, such as silt, sand and gravel, in flowing water occurs under a variety of natural and man-made conditions. It determines the evolution of riverbeds, estuaries and cost lines, therefore consequently affects the formation and stratification of the bed surface level. It is necessary to investigate deeply the mechanics of sediment transport because it affects the functioning of many hydraulic structures and determines their lifetime. The mechanics of sediment transport, quantitatively and universally applicable laws governing the motion of the transporting fluid and the transported sediment, remains remarkably in lack of knowledge. This lack of accepted fundamental principles might explain why the literature on sediment transport consists of a lot of research papers.

Open channel flow over movable boundaries behaves differently from open channel with rigid boundaries. In alluvial channels, rigid boundary relations apply only if there is no movement of bed and bank material. Once the general movement of the bed material has started, the flow and boundary interact in a complex manner. To apply an analytical approach to such problems of river development and natural alluvial channels is both difficult and time consuming, because of the complexity of the processes occurring in natural flows involving the erosion and deposition of transported material. Generally, most of the relationships describing the morphological processes in alluvial channel systems have been derived empirically. Nevertheless, if a greater understanding of the principles governing these processes is to be developed, such empirically derived relations and formulae must be incorporated in their proper context.

Cooper et al (1972) presented a study concerned with description of the nature and the scope of existing collection of experimental data, concerned individually and as a whole. The study also compared the scope of experimental conditions with conditions likely to be encountered in engineering practice. Bogardi (1965) examined the various criteria defining the regime of sediment transportation used in Europe, including the sediment transporting capacity and the stage of the incipient motion. A brief review of American results was given in comparison to the European studies. The main conclusion was that almost every problem connected with the regime of sediment movement may approached through use of the same set of interrelated variables. Bogardi classified sediment studies into two groups; those conducted in connection with the sediment transportation of a given natural watercourse, and those related to sediment movement considered as a physical phenomenon.

The transport of non-cohesive sediment in an open channel is a complex process, and the physics of this two-phase motion is incompletely understood. Furthermore, the competent condition characterizing the incipient motion of sediment particles was realized as a condition of importance. It is often termed as the “critical condition”. By definition, when conditions governing the sediment movement exceed this critical condition, the sediment particles start moving and the plane bed is disturbed. An early incipient-motion investigation was that of Shields in 1936 establishing his well-known dimensionless graphical relationship.

Since 1950 increasing use has been made of dimensionless notation in sediment research. Bogardi (1965) presented that; it *is often found that the same dimensionless numbers have different designations in individual investigations. Ackers and White (1973) mentioned that, many theories have put forward in attempts to provide framework for the analysis of the data on sediment transport. Some of these theories based on the physics of particle motion and others on similarity principles or dimensional arguments. They developed and examined a dimensionless based framework for the analysis of sediment transport data in which the advantage was shown and the physical arguments are used in deriving the form of the tested functions.*

Kuiper (1960) summarized the essential process in the formation of a river delta. The trap efficiency has been investigated and presented by many researchers. Moore et al (1960) reviewed and summarized available information pertaining to trap efficiency and made it available in a unified form. Brune in 1953 pointed out that, the trap efficiency of a reservoir depends on a number of factors. Among these are the ratio between the storage capacity and inflow, age of the reservoir, shape of the basin, the type of the outlets and method of operation, the size grading of the sediment and the behavior of the finer sediment fractions under various conditions.

The ratio between storage capacity and inflow has been expressed in a general way by the capacity-watershed ratio. In 1943, Brown developed a curve relating the capacity-watershed ratio and trap efficiency. Churchill in 1948 took into account both the detention time and velocity of flow through the reservoir. He developed a “sedimentation index” which represents the period of detention divided by mean velocity. His curve relates the trap efficiency to the sedimentation index. In Brune’s study, a thorough search was made for all reliable records of reservoir trap efficiency. He pointed that, it is possible to study trap efficiency by a number of different methods. Brune found that, it is much better to use the capacity-inflow ratio than using the capacity-watershed ratio.

Most of the artificial storage is provided in the form of surface reservoir. Einstein (1961), regarding needs in sedimentation, mentioned that reservoirs change the rivers regime, particularly with respect to their sediment characteristics. He mentioned that any large storage reservoir permanently stores the entire sediment load of the stream. In addition, part of the sediment storage occurs in the storage volume and part in the channel bottom upstream by backwater effects. Sediment survey of reservoirs is an important issue. It should include information on the volume-weight of deposited sediment. This unit of measurement provides means of determining the sediment yield of a watershed. A detailed study was made of the volume-weight of deposited sediment in Kansas, in 1960. Heinemann (1962) showed that the volume-weight of sediment primarily depends on the clay fraction of the sediment and, to a much smaller extent, on the depth of the sediment in a sediment deposit. The clay fraction tends to vary-inversely with distance upstream from the dam.

Mao and Rice (1963) developed a procedure to provide a means of evaluating the need for sediment control in canals. The procedure utilizes basic concepts of the Einstein bed load function to evaluate sediment concentrations and size distributions entering the head reach. The sediment transport capability of an erodible channel can be related to the concentration and size-distribution characteristics of the sediment delivered to the channel. Toffaleti (1969) presented a comparison of computed versus measured sediment loads that covered a wide range of conditions, to show his proposed procedure for an analytic determination of sand transport. The procedure was adapted to computer programming.

Parker (1996) mentioned, in his work concerning the interaction between the basic research and applied engineering, to turbidity currents and debris flows in oceans. Oceanic turbidity currents, or dense turbulent underflows laden with sediment are major mechanisms for the delivery of sediment from the continental shelf across the continental slope and into deep water. Others indicated that oscillations cause sediment movement and foreshore profile change together wave induced currents. A flow-sediment model can serve as a predictive tool for quantitative analyses of the impact of costal line configurations on flow and suspended sediment concentrations in costal waters.

Generally, research work on sediment transport is not easy to be completely surveyed because of the huge amount of the literature published on the topic, which is relevant to some other fields of engineering, {E1 - E15}. Thus, spotlight on some works is the reasonable way to be used. Some of such works were carried by Henderson (1961), Zernial and Laursen (1963), Nordin (1964), Ansely (1965), Sayre (1969), Shen (1971, 1972), Graf (1971), Willis et al (1972), Yalin (1972), Szechwycz and Qureshi (1973), Navak and Nalluri (1975), Kao (1977), Parker and Anderson (1977), Pazis and Graf (1977), Chang (1979), Focsa (1980), Brownlie (1983), Wang (1984), Park and Jain (1986), Tywoniuk (1972), Billi et al (1992), Lai (1996), Osman and Sharfi (1999), etc.

2-1-1 AGGRADATION AND DEGRADATION PROCESSES

The mechanics of aggradation and degradation processes for sediment of uniform size have studied by many investigators in the last decades. Some efforts investigating this sector include Hannad (1972), Jaramillo and Jain (1984), Jain and Park (1989). Newton, in 1951 conducted a series of degradation experiments using uniform sediment and found that the bed elevation and bed slope decreased asymptotically with time. Bhamidipaty and Sher, in 1971, concluded from the analysis of Newton data and experimental data that the bed elevation in a degradation channel decreases exponentially with time. Yen et al (1992) performed a sense of over loading experiments with uniform coarse sediment and found that both the aggradation wave speed and the mean sediment transport velocity increase with the initial equilibrium bed slope.

For non-uniform sediment transport with respect to aggradation and degradation, little and Mayer (1976) performed a series of degradation experiments. They focused on the variation in sediment gradation of the bed surface during the armoring processes. Ribberink, in 1983 experimentally studied the vertical sorting phenomenon of sediment having an idealized gradation under equilibrium conditions and proposed a transport Layer concept. Wilcock and Southard, in 1989, investigated the interaction between bed surface texture and bed configuration by making careful measurements and observations in the startup and equilibrium states in a re-circulating laboratory flume.

Yen et al (1992) investigated the behavior of channel bed evolution under the condition of overloading followed by under loading of highly non-uniform sediment in a laboratory flume and developed a linear model of sediment transport. The coefficient involved in the model was evaluated from the experimental results, and a recovery ratio concept of riverbed was developed. The conclusion of the research work, due to the effects of hydraulic sorting and armoring, the aggradation-degradation cycle in the alluvial channel composed of non-uniform bed material is irreversible.

The streambed may be aggrading, or not changing, that is to be in equilibrium, in a specific reach. Factors that affect bed elevation changes are various, some of them; dams and reservoirs, change in the water shed land use, canalization, changes in the down stream base level of the reach, etc. Different approaches have been suggested to evaluate aggradation. Komura and Simons (1967) proposed method of evaluation of bed degradation. In addition, Aksoy,in 1970, and Ashida and Michiue, in 1971, suggested a formulae to predict degradation, the later took armoring into consideration.

Simons and Senturk (1992) classified Aggradation and degradation into four types:

1. Long-term aggradation or degradation.
2. General scour and contraction scour.
3. Local scour.
4. Lateral shifting of the stream.

2-1-2 ARMORING PROCESSES

Bed armoring, measured as the ratio of surface to subsurface bed material grain size, is highly variable especially in a channel in which past and present bed aggradation indicates that the sediment supply has exceeded the transport capacity. As the flow varies over a channel with non-uniform bed topography, local variability occurs in both the direction of sediment transport and the magnitude of the boundary shear stress. This makes some parts of the channel continue to supply sediment to be transported, while others become sites of deposition. The degree of armoring is the coarseness of the armor layer relative to that of the underlying bed material. Armoring occurs on a stream when the forces of the bed during a particular flood are unable to move the larger sizes of the bed material. When armoring occurs, the coarser bed material will tend to remain in place. This armoring effect can decrease scour depths, which were predicted to occur based on formulae developed for sand and fine material for particular flow conditions. Research work on armoring studies and modeling include Bary and Church (1980), Lee and Odgaard (1986), Codell et al (1990), Jain (1990).

An exact mathematical approach to the problem is difficult. The fine particles are hiden underneath the coarse particles forming the surface layer and the shear applied by the flow to these particles is smaller than the shear taken by the coarse material. The initiation of motion is basically different when the bed material is non-uniform instead of being uniform. Egiazaroff (1965) modified shields' criterion of beginning of motion and obtained the non-uniform material is less mobile than the corresponding uniform material. Garde et al (1977) and Gessler (1990) considered the probability of the removal of particles by transport and determined the resulting median diameter of the remaining particles forming the armor coat. Knoroz, in 1971, Suggested a formula for describing the conditions under which natural armoring occurs.

Karim and Kennedy (1982) quantified armoring as the fraction of bed surface covered by the immobile sediment particles and is expressed as a function of time by calculating the volume of immobile sediment exposed as the bed degrades, and then determining the fraction of the bed surface it will cover. It is assumed that sediment discharge is reduced in direct proportion to the fraction of the bed surface area that is armored, and the flow resistance approximated by a fixed-bed friction factor relation.

Land use changes in watersheds frequently cause changes in the size distribution of riverbed substrate and the degree of armoring in bed of rivers. In large part the size of armor layer controls the initiation of sediment transport and gravel mobility. The spatial variability of streambed armoring is high, and the stream-wise sorting of bed particles during recessional flows can affect size selection in bed load transport during subsequent rising stages. The role of non-uniformity and unsteadiness of flow in natural channels and its influence on channel armoring and fine sediment deposition in coarse gravel bedded rivers should be addressed. Elsadig M. Abdalla et al (1986) mentioned three different types of bed armoring formations. These are: firstly, self-armoring, which is the selective transport of the solid materials eroded from bed surface such that only the larger particles will remain at the bed surface. Secondly, the exterior armoring, which is the deposition of larger particles on a bed of smaller ones. Thirdly, intermediate armoring, which is a combination of the two types mentioned above.

The lack of data on the rate at which armoring effects reduce sedimentation rates is the major obstacle to predict the amount of sediment accumulated at a particular site. To make an estimation of the total sediment input at a certain reach with confidence, it is necessary to measure the armoring process in the field.

2-1-3 MODELING OF MORPHOLOGICAL PROCESSES

River morphology and mechanics are vitally concerned with, because the information makes possible interpretation of the previous sediment transport of the river. In addition, other studies related to reservoir predictions such as water quality and thermal stratification are of much interest to authors of hydraulic and environmental researches like James (1984), Karpik and Raithby (1990), etc. The investigation and modeling of reservoirs, rivers and channels, response to changes and variable events of evolution, provides information and insight into the long-term of the alluvial system adjustment to alter the hydrologic and hydraulic conditions.

A lot of research work has been carried to investigate and predict the river response to change. Lane, in 1955, studied the changes in river morphology in response to varying water and sediment discharges. Similarly Leopold and Maddock, in 1953, Schumm, in1971, Komura and Simons (1967) have investigated channel response to natural and imposed changes. Simons et al, in 1975 developed a useful relation for predicting system response establishing proportionality between bed material transport and several related parameters. Quantitative field studies of morphological processes are rendered difficult by site-specific factors such as hydrologic regime, different types of sediment brought by tributaries, bed level variation and scale of the problem under consideration, which may vary from several hundred meters to several hundred kilometers. Researchers have resorted the techniques, which allow for isolating the effects of various parameters.

Several numerical models have been developed to study various morphological processes in alluvial channels. Soni et al (1980) conducted laboratory experiments that covered a wider range of flow and loading conditions, and developed a mathematical model for aggradation in a long channel. Problems such as bed degradation and armoring downstream of dams, reservoir sedimentation, and hydraulic sorting were thoroughly studied empirically and numerically by many authors. Ahmed (1994), Lu and Shen (1986), Karim and Kennedy (1982), Park and Jain (1987) and Rahuel et al (1989) suggested different models to treat sediment transport and hydraulic sorting of sediment mixtures present in alluvial channels.

2-2 ROUGHNESS AND FLOW RESISTANCE

There is a great utility in expressing resistance to steady fully developed flow in open channels in terms of a dimensionless quantity, the friction factor, which depends on the Reynolds number and bed roughness. It worth mentioning that, the problem of defining roughness in alluvial channels dates back so many years. An analysis of flow in alluvial channel is extremely complex because of the many variables involved and the difficulty of measuring them. Many problems are present with regard to friction factors because when the material forming the channel boundaries moves, the form of the bed roughness is molded by the flow, and the friction factor must be evaluated separately for each open channel flow problem. A survey on early empirical studies concerning the resistance to flow in fixed-bed open channel has been presented by the Task Force on friction factors in open channels of the committee on hydromechanics of the hydraulics division of the ASCE in 1963.

In 1768, Antione Chezy reasoned that the resistance would vary the wetted perimeter with the square of the velocity, and that the force to balance this resistance would vary with the cross-sectional area of the flow and with the slope. Chezy's manuscript was not published until 1879, but his method gradually became known and Chezy's coefficient came to present. The first systematic and extensive effort to discover how this coefficient varies under different conditions was begun by Darcy in 1855 and continued by Basin, in 1865, who proposed a formula based on wall roughness. In 1869, Ganguillet and Kutter published their well-known formula. In 1881, Hagen came to the same conclusion by a least-squares study of the same data as Ganguillet and Kutter had used.

In 1889, Manning mentioned the same result but recommended a new roughness coefficient and later his formula became in more use. Strickler, in 1923, reported that for streams whose beds are composed of cobbles or small boulders, the Manning's coefficient is a function of the average sediment diameter. In 1938, Keulegan developed a formula for open channel resistance based on Von Karman's constant. He made use of what is called the friction velocity. In 1948, Bretting suggested three exponential equations each approximating the flow equation for a particular range of values of relative roughness.

Basically, two superior theories were recognized in the design of uniform open channel. These are the regime theory and the tractive force theory. The regime theory initiated by Kennedy in 1895, when he produced his classic equation and it was applied extensively in design as originally presented. In 1914, Gilbert experimentally showed that many configurations of bed roughness could be formed by the flow and that the resistance factor varied with the bed form. In 1919, Lindley introduced other regime equation by correlating data observed in canal surveys. He was the first to introduce bed width and depth as regime variables. Other equations developed by other investigators, such as Lacey in 1927 and Bose in 1936.

Inadequacy of the regime method mentioned by Simons and Albertson (1960). One disadvantage is that, it has not been developed based on the wide variety of conditions encountered in practice. Also the theory fails to recognize the important influence of sediment discharge on design. The tractive force theory was formulated on the basis of stability of bank and bed material as a function of their ability to resist erosion resulting from the drag force exerted on them by the moving water. This concept has been widely applied in sediment transport but only to a limited extent in connection with design of channels in alluvial materials. Use of this method of design, has been suggested by Schoklisch in 1937.

In 1950, Einstein presented two relations for solving the problem of resistance to flow in channels with movable boundaries and bed forms. The first relation applies to the resistance due to grain roughness and the second define the resistance to flow caused by bed forms. His method concludes that the factors governing the flow resistance are the Froude number and the relative roughness. In 1952, Einstein and Barbarossa determined from both flume and field data that the Einstein form resistance was a function of sediment transport rate. Einstein and Barbarossa assumed that the bed shear stress is made of two components; the first is shear stress from intrinsic resistance due to the sediment itself, and the second is shear stress due to bed form resistance.

The variations of the Manning's roughness coefficient or Darcy's friction factor for two series of sands and various bed forms given by Simons and Richardson (1960). The given values agree with resistance coefficients given by Brooks and by Laursen in 1958, and by Kennedy in 1961. Shen, in 1962, defined the resistance variation through a parameter that varies with the particles Reynolds number. Engelund (1966) suggested an expression for the friction loss due to bed forms of certain wavelength with an expansion-loss equation. In 1967, Engelund and Hansen proposed a graphical relation from which stage-discharge relationship can be determined and shows the lower and upper regimes and the transition between them. Simons and Richardson, in 1967, suggested particular formulae for each bed form instead of the previous relations that describe the resistance as a whole. In addition, Simons and Senturk (1992) gave a group of formulae defining the resistance to flow in the transition regions that also indicate the bed forms. They mentioned also the suggestion of Senturk, in 1969, which propose a parameter for resistance to flow on a movable bed and accordingly developed formulae predicting not only the bed resistance but also the formation of bed configurations

Lovera and Kennedy, in 1969, found from data collected on movable plane beds that the skin resistance was a function of the Reynolds number and the relative roughness. Alam and Kennedy, in 1969, calculated the form drag as the difference between the total resistance and the skin resistance, and it is a function of the flow Froude number and the relative roughness. They investigated a series of flume and field data, and suggested a functional relation concerning graphical prediction of the bed form friction factor. Raudkivi (1967), Vanoni and

Hwang (1967), Engel and Lau (1980) investigated the relationships between bed forms and flows under controlled laboratory conditions. Garde and Rajin, in 1985, also summarized several studies that also include their own studies. A number of investigators have used numerical models to compute the form friction factor as a function of bed form height, bed form length and flow depth.

The physical processes that determine resistance to flow in natural streams vary widely depending upon the character of the stream. Various researchers explained such morphological processes in different ways. Meyer-Peter and Muller, in 1948, based on so many experimental works, assumed that the energy slope is a characteristic of the interaction between solid and liquid motion of a sediment-laden flow. A given portion of the energy is consumed for solid transport and the remaining for liquid motion. While Einstein, based on stochastic approach, tried to determine the amount of shear stresses consumed by the solid particles in their movement. Simons and senturk (1992) suggested that the friction slope can be separated into two parts; part required to overcome surface drag and part required to overcome form drag.

Resistance to flow in alluvial channels varies between wide limits and the form of the bed roughness is a function of fluid properties, flow and sediment characteristics and channel geometry. This sector met great attention of investigators. Van Rijn (1984) and Karim (1995) reported studies relating bed resistance directly from the bed form geometry. Chiew (1991) investigated the importance of bed armoring on bed resistance. Lyn’s (1991) involves sediment-laden flow under bed conditions. Raju et al (1998) studied the resistance of coarse sediment beds. The power principles have been applied to interpret the development of the bed forms and proposed a single diagram for determining flow resistance under various flow conditions. Yu and Lim (2003) investigated the bed resistance of two-dimensional flows over bed forms. Some of these investigations are those of Burkham and Rouse (1965), Yen et al (1972), Dawdy (1976), Senturk (1978), Knight and Mc Donald (1979), Van Rijn (1984), Shen et al (1990), Knight and Brown (2001), {E16 – E18}, etc.

2-2-1 FLOW RESISTANCE FORMULAE

For alluvial channels, the shape of the channel depends on the type of boundary material comprising bed and banks, the channel alignment and the magnitude of hydraulic variables including shear stress distribution, velocity distribution and stream power. Simple formula to compute flow velocity in a canal has been developed by Chezy, which is based on the assumption that the drag force may be expressed in terms of the dynamic variables of resistance, viscosity and velocity. Later, Manning developed his relation with his roughness coefficient, which is investigated by Strickler and other researchers as mentioned before.

The concept of semi-logarithmic formulae was attempted to apply to open channels by different researchers such as Darcy-Weisbach. Generally, the Darcy-Weisbach friction factor is a function of the relative roughness, Reynolds number, and the shape of cross section. Extensive efforts have been performed to determine the friction factor, Darcy-Weisbach parameter, to study the resistance to flow in three categories:

- Open channel with fully developed roughness
- Hydraulically smooth channels
- Natural rough boundaries in a transitional zone

Many resistance formulae have developed in form suggested by Simons and senturk (1992), which relates the dimensionless velocity to the relative roughness, Froude number of sediment particle and Reynolds number for both the flow and the sediment particle. It was differentiated between two forms of friction in alluvial channels. The first is the surface friction, which occurs when a channel with a plane bed is subjected to turbulent flow. The second one is the form friction, which corresponds to the occurrence of bed forms.

Recently, Yu and Lim (2003) proposed a modified flow formula for flow in prediction in alluvial channels. The formula starts with the conventional practical method to calculate the flow velocity, which is based on Manning equation. This is supposed to be applicable, as mentioned, to flat bed channel with non-erodible material or loose bed material such that the flow strength is too weak to dislodge the granular particles. Yu and Lim mentioned that, once the flow rate is high enough to dislodge the sediment particles, it is assumed that, bed forms would develop on the bed and the bed resistance would increase. In this case, Manning coefficient is no longer valid since it would vary not only with sediment size, as by Strickler's, but also the flow rate and flow depth. That is justified by increase of flow resistance in the latter.

To express surface and form friction, two general approaches are used as mentioned by Simons and Senturk (1992):

1- The idea developed by Meyer-Peter and Muller, in which it is assumed that a certain fraction of the energy is consumed to overcome the resistance due the surface friction, and the remainder is used to overcome the form friction.
2- The concept given by Einstein, in which the hydraulic radius is considered to represent the volume of rectangular prism of a unity base with a height equal to the water depth, considering the fact that the hydraulic radius increases for increasing values of boundary resistance.

In order to accurately model open channel hydraulic conditions over a range of discharges, the variability in resistance to flow and velocity must be evaluated. Several approaches for selection of values of resistance to flow were investigated by a number of researches for a range of channel types. Table (2.2.1) shows a list of some Resistance formulae;

Table (2.2.1) Some Resistance Formulae

Author	Date	Formula
Chezy	1769	$U = C\sqrt{gRS_f}$
Manning	1889	$U = \frac{1}{n} R^{2/3} \sqrt{S_f}$
Darcy-Weisbach	1938	$\frac{1}{\sqrt{f}} = C \log(a \frac{R}{k_s})$
Stickler-Meyer-Peter	1948	$n = \frac{D_{90}^{1/6}}{26}$
Simon and Richardson	1967	$\left(C / \sqrt{g}\right) = 5.9 \log(d / D_{85}) + 5.44$
Senturk	1973	$\frac{U}{U^*} = 6 - 2\log C + 6.5 \log \frac{R}{D_{65}}$
Simon et al	1989	$\sqrt{8/f} = 0.89(\frac{d}{D_{84}})^{0.45} (\frac{D_{84}}{D_{50}})^{0.87} \sqrt{S_f}$
Yu and Lim	2003	$U = 6.7\sqrt{gD_{50}S_f} (\frac{R}{D_{50}})^{2/3}$

2-2-2 CHARACTERISTICS OF CHANNEL ROUGHNESS

The factors affecting resistance to flow in mobile-bed channels are many and quite complex. Flow resistance is usually due to a combination of skin friction form drag and water surface losses. To account for the variability in resistance to flow some simple approaches can be utilized. For large rivers with fine sand bed, an exponential decay-type relationship can be found to relate the roughness coefficient and the discharge. On the other hand, for coarser sand-bed rivers with some gravel, a grain size distribution is usually used to obtain a similar relation. Such relations were fitted to some alluvial channel models utilizing an iterative procedure to compute water surface profile over a range of flows accounting for the variable resistance to flow.

Resistance to flow is categorized according to the roughness scale by number of researchers such as Herbich and Shulits (1964), {E19 – E20}. The division of scales is based on the concept of relative submergence of the bed material of the channel. The relative submergence is defined as the ratio of the flow depth to the representative particle size, as described by Bathurst (1978). Small-scale roughness occurs at high relative submergence where the flow resistance is due primarily to skin friction. Large-scale roughness occurs at low relative submergence where the form drag around the individual particles and distortion of the free surface are the dominant processes. Large-scale roughness occurs when the particle size is on the same order as the flow depth. Intermediate scale roughness is a transition region between small and large-scale roughness. In this region, the particles are sufficiently large to cause disturbance of the water surface.

Large-scale roughness occurs when relative submergence using the D_{84} size as the characteristic dimension is less than 1.2. Similarly, small-scale roughness occurs when the relative submergence is greater than 4.0, and intermediate scale for relative submergence between 1.2 and 4.0. Alluvial channel with plane boundaries the same concepts of roughness height of rigid bed geometry could be applied when the flow remain below the threshold of particle movement.

The flow resistance of small-scale roughness can be described using the boundary layer theory, the approach that requires the roughness elements on the boundary act collectively as one surface, applying a frictional shear on the flow, hence the shear is translated into a velocity profile. The shape of the profile is determined by both the roughness and the channel geometry. For large-scale roughness, the velocity profile is completely disturbed since the roughness elements act individually, producing a total resistance based mainly on the roughness and to some extent on the channel geometry.

2-2-3 BED FORMS IN ALLUVIAL CHANNELS

It is known that there is an interrelationship between the friction factor, and sediment transport rate, channel bed configuration, and channel geometry, when considering the flow in an alluvial channel. The interrelationship between the flow and the bed material and the interdependency among the variables makes the analysis of flow in alluvial channels is extremely complex. However, all problems those occur in alluvial rivers and channels can be analyzed and solved with the knowledge of different types of bed forms, the flow resistance and sediment transport associated with each bed form and how the various variables such as flow depth, slope and viscosity, etc. affect the bed form. A lot of researches touched bed forms in alluvial channels and their modeling such as Khann (1970), Gill (1971), Pratt (1973), Song (1983), {E21}, etc.

The bed forms generated on the bed of an alluvial channel by the flow were clearly described by Simons and Senturk (1992). The bed configurations that may form in alluvial channel are plane bed without sediment movement, ripples, dunes, plane bed with sediment movement, anti-dunes, and chutes and pools. These bed forms are listed in their order of occurrence with increasing values of stream power or similar parameters for bed materials. The variations of the different variables with flow regimes and bed forms for various sand sizes were investigated by many researchers. Work on the prediction of bed forms has involved both theoretical and empirical approaches.

In the absence of universally acceptable analytical solutions for the prediction of bed forms, some researchers have tried to fill the gap by presenting dimensional and non-dimensional plots based on flume data supported by some data from natural channels. Based on extensive flume and canal data, Simons and Richardson (1961) presented three different methods to analyze bed form roughness. Firstly, the entire analysis is based on the principle that the energy loss due to grain roughness can be estimated from a logarithmic roughness relationship. Secondly, the analysis based on dividing the flow depth into two parts and taking in consideration the increase in energy dissipation due to form roughness. Thirdly, based on development of velocity correction to be a function of the hydraulic radius and energy slope. Generally, analysis of bed form behavior must take into account the relationship between three separate factors. These are the following:

1- The shape of the bed form profile.
2- The flow of the fluid over the bed form.
3- The sediment transport over the bed form.

These three factors give rise to three separate relationships, namely,

1- The relationship between the sediment transport and the fluid flow over the bed form.
2- The relationship between the fluid flow and the bed form profile.
3- The relationship between the bed form profile and the sediment transport.

The first relationship is basically a bed-load transport relationship such as have been proposed by many researchers. Regarding the second relationship, essentially all mathematical models have made use of two-dimensional potential flow to obtain this relationship. The third relationship represents the continuity of sediment motion. It worth mentioning that, Study of the properties of bed forms is an important factor in sediment transport predictions, that is to understand how bed forms are related to the characteristics of fluid, flow and bed material. Another reason is to understand how bed forms affect the resistance to flow and sediment transport in alluvial channel.

2-2-4 FLOW REGIMES

Simons and Richardson (1961) divided the flow in a sand-bed river into two flow regimes separated with a transition zone. Each of these two flow regimes is characterized by similarities in the shape of the bed forms, mode of sediment transport, process of energy dissipation and phase relation between the bed and water surface. The various flow regimes are classified as lower regime, upper regime and the transition zone in which bed configurations range from dunes to plane bed or to anti-dunes.

The lower regime is associated with, bed configurations, ripples and dunes. This regime begins with the beginning of motion. The resistance to flow is large and sediment transport rate is small. Water undulations, if exists, are out of phase with the bed surface, and there is a relatively large separation zone downstream the crest of each ripple or dune. Resistance to flow is caused mainly by the form drag. In upper flow regime, plane bed and anti-dunes are the associated bed form, resistance to flow is relatively small and sediment transport rate is relatively large. The water surface is in phase with the bed surface and normally the fluid does not separate from the boundary. Resistance to flow is a result of the grain roughness with the grains moving and the energy dissipation.

2-3 HYDRAULICS OF ALLUVIAL CHANNELS

Recently, many research projects came to the fore in order to rationally develop and properly manage water resources. Research work on sediment transport started in nineteenth century. Most of researches at that time were based on experimental work. Many researchers have investigated the transport of sediment, in both rigid and mobile bed. Many of the formulas dealing with sediment transport in mobile bed channels are empirical or semi-empirical in nature and have been based on the results of laboratory flume experiments. Where the collected experimental data used to develop a formula cover only a narrow range of the flow

conditions, there exists a danger of errors resulting when the formula is extrapolated to practical engineering conditions. Furthermore, the state of the knowledge of fluid mechanics and engineering hydraulics have progressed to the point that, it is possible to predict with accuracy adequate for most practical requirements the hydraulic roughness of fixed geometric channels. For flow in alluvial channels, the main difficulty is that the channel geometrical characteristics and hence the hydraulic roughness depend on the flow depth, the velocity and the sediment transport rate. Consequently, to make depth-discharge predictions for alluvial streams, additional information is required.

Einstein and Barbarossa, in 1952, were the first to develop a depth-discharge predictor. They proposed that the cross-sectional area and the hydraulic radius of the channel each to be treated as consisting of two additive parts. In 1956, Bagnold proposed a theory for sediment transport and bed forms development based upon his theoretical and experimental studies of the behavior of grain dispersion under the action of the shear stress. The theory extended from the plane bed equilibrium condition to explain the development of the bed features and the contribution of their associated form drag in the reestablishment of the stress equilibrium at the bed surface. In 1962, Shen tried to improve the Einstein and Barbarossa method and in particular to extend it to materials other than sand. Grade and Raju (1966) collected a large record of laboratory and field data obtained by other investigators to evaluate the earlier depth-discharge relations proposed by Einstein and Barbarossa and that suggested by Shen. They found that none of these methods is particularly reliable and then proceed to analyze the data led to conclude that an equation of the Manning form with a variable coefficient should be adapted.

As a part of alluvial hydraulics, Lacy published his “regime theory” of the design of stable channels in incoherent granular material. He presented equations to describe channels in which the bed was live but stable, that is, the sediment load was being supplied from upstream at a rate sufficient to balance any scour due to bed movement. In this condition, the channel is to be “ in regime”. On the other hand, early geologists of the U.S. Geological Survey, who worked with rivers of mobile boundary and tried to find the quantitative system behind their self-adjusted dimension, formed the quantitative geomorphic theory.

Blench (1969) presented the importance of elaboration of a general theory of a riverbed development and sediment transport in a stream. He stated that, the regime theory school already provided a scientific quantitative base for practically most important phase of canal transport of sand, with non-rigorous practical extensions that deal usefully with almost engineering problems of sand rivers with small bed-load discharges and large gravel rivers. The geomorphic theory also made non-rigorous practical quantitative extensions to sand rivers, and deals with large discharges and possible further extension to gravel. Each theory complements and benefits from the basic practical findings of the other; both aimed to eventual improvement of the basics of the understanding of sediment transport. Blench concentrated also on two important general points became obvious in mobile bed hydraulics. These are the degrees of freedom and the phases. On the other hand, different visible phases should be expected to require different forms of detailed equations.

It has been noticed that once the critical condition is reached on the bed of an alluvial stream, the individual sediment particles on the bed starts moving thus disturbing the initial plane bed. With changing flow conditions, the bed and the water surface take various forms. Earlier researchers discovered that the regime of flow has a great influence on such factors as resistance to flow and the rate of sediment transport. Simons et al (1962) clarified the effect of changing bed and water surface characteristics on the nature of stage discharge curves. Based on flume data and some data from natural stream, Garde and Raju (1963) proposed a few criteria for prediction of the regime of flow. They concluded that, the shear stress is not always an effective parameter for satisfactorily predicting regimes of flow. Other criteria based on dimensional analysis can be used.

The stability of alluvial channels is of considerable importance in irrigation schemes, river improvement and similar hydraulic projects. The majority of the relevant laboratory studies have been conducted in rigid-walled flumes having a bed of mobile solid particles. Acker's investigation, in 1963, has been made under completely free boundary conditions. This experiment provided a basis for a review of the theories on channel stability and sediment transport with reference made particularly to the two major basis of thought on the topic. Namely, the physical approach based on a consideration of the forces on or movement of individual grains. Secondly, the regime concept developed from the analysis of data from stable canals. The extensive investigation of Acker (1964) showed that the empirical equations for stream geometry applicable to the system of channels are entirely consistent with those deduced by the combination of three physical relationships, namely, the resistance formula, the sediment transport function and the width to depth ratio. In 1965, Yalin stated that, if the shape of the cross section of the flow, the shape of the particles of the bed material and the shape of the particle-size distribution curve of the bed material are specified and if the values of kinematic viscosity of water, the fluid density, the diameter of a typical sediment particle and its density, the flow depth and the energy gradient are known, then a steady and uniform two-phase phenomenon may be defined. Formerly, in 1958, Brooks found that, in the laboratory flume, neither the velocity nor the sediment discharge concentration could be expressed as a single valued function of the bed shear stress, or any combination of depth and slope, or hydraulic radius and slope.

Maddock (1973) stated that, Yalin statement applies to a situation in which width is not fixed, where water and sediment discharges are independent variables and velocity, depth and slope are the dependent variables, whereas Brooks statement is correct for flumes having a constant width in which discharge and depth are independent variables and sediment discharge and slope are the dependent variables. He also mentioned that, to establish determinate relations for the solution of the dual problem of resistance to flow and sediment transport in alluvial channels is an impossible task, however, relations can be fond that will permit the prediction of some of the odd or seemingly irrational and inconsistent behavior of alluvial channels.

In 1967, Sayre and Conover developed a general two-dimensional stochastic model for the dispersion of sediments. In 1968, Yang used a similar approach to develop a one-dimensional stochastic model for sediment movement. Hill et al (1969) proposed a method to predict the occurrence of the different bed forms on a bed that is initially flat. The mechanism of particle motion in a channel was described Grigg (1970). He mentioned that, the motion of each particle consists of a series of alternating steps and rest periods and to describe the particle motion completely, it is necessary to specify certain probability distribution. His study was limited to the ripples and dunes bed form of lower regime.

Rana et al (1973) stated that, transport of sediment alter the size of sediment particles by abrasion and by sorting. Abrasion is the reduction in size of particles by mechanical processes such as grinding, impact and rubbing, while sorting is a result of differential transport of particles of different sizes. Rana et al also mentioned that, in historical handling of this phenomenon, starting with Sternberg in 1875, the abrasion of sediment during the transport was considered to be the major factor responsible for sediment size reduction in alluvial rivers. As a result of experiments carried by later investigators, it was realized that the rate of size reduction by abrasion is too small to account for the magnitude observed in nature.

In 1963, Kennedy and Brooks outlined concisely several sets of independent and dependent variables that determine the behavior of alluvial channel flows. Vanoni (1974) mentioned in such set for flow the independent variables are the fluid properties, bed sediment properties, and mean velocity, flow depth and flow width, whereas the dependent variables of the set are the water discharge, sediment discharge, hydraulic radius slope and the friction factor. In order to solve for the dependent variables, five relations are needed. These are the continuity equation, the relation for the hydraulic radius in terms of the flow depth and width, a relation giving the slope in terms of the independent variables, a sediment transport relation and a friction factor relation. Several workers arrived to the list of pertinent variables first proposed by Kennedy and Brooks.

A lot of the literature sources mentioned that numerous investigators have published predictors for irrigation canals and river flow profiles. Ahmed (1992) indicated to the adequacy of irrigation canal and proposed a calibration technique for canal design criteria. The flow surface profiles computations include Ligget (1961), Chen and Wang (1969), Eichert (1970), Gill (1971), Mc Bean and Perkins (1975), etc. Shimizu and Itakura (1989) mentioned the several studies have been made to evaluate flow and bed variation. Studies that have been carried out among others are Engelund, in 1974, Falcon and Kennedy, in 1983, Struiksma et al, in 1985, Odgaard, in 1986, Ikeda et al, in 1987. Wormleaton (2004) investigated large-scale physical model with flood planes using graded sediment in a meandering channel. Other similar investigations are {E22 - E23}, Benson et al (2001), Knight and Brown (2001), Sellin et al (2003) and Neyshabouri (2003).

2-3-1 BASIC CONCEPTS OF CHANNEL FLOW

Fluid flow is generally classified according to several bases, which apply to both conduit flow and open channel flow. Time as a basis classifies the flow either steady or unsteady. Distance as a base classifies the flow as uniform or non-uniform. Uniform flow is the exception rather than the rule in open channel flow. Manning equation is an accepted relationship between the pertinent variables; it is used to establish what is termed the normal depth for uniform flow in a channel. Usually, the depth of flow changes to compensate for something which makes the flow non-uniform; such things are bends, contractions, expansions, obstructions, changes in channel roughness, changes in channel slope and changes in channel cross section. When the non-uniform flow is gradually varied, Manning equation is still considered by many researchers to give a reasonable measure of the boundary resistance. On the other hand, internal stability as a basis classifies the flow as laminar or turbulent. For open channel flow, there are also separated bases which in use. The water wave as a ratio of the internal forces to gravitational forces, the dimensionless parameter Froude number, classifies the flow as sub-critical, critical, or supercritical flow.

When an interface exists between two fluids such as air and water, the fluid property called the surface tension will have a bearing on the flow. The Weber number expresses the relative effect of this property. Also in open channel flow, there is no physical boundary to control the flow at the free surface. As a consequence, the free surface in non-uniform flow distorts in such a manner as to establish a force regime, which brings about a state of equilibrium; that is, the flow is steady but non-uniform. Investigations on open channel hydraulic process and modeling are available in so many sources such as Whittington (1963), Chiu et al (1976), Jarrett (1984), etc. in addition to the large amount of textbooks concerning the subject. It worth mentioning, Hydraulic properties of natural open channels are generally very irregular. In some cases empirical assumptions consistent with actual observations and experience may be made such that the conditions of flow in open channels become amenable to the analytical and numerical treatment of theoretical hydraulics. The following paragraphs give some basic principles in open channel flow

For various flow situations, knowledge of the qualitative behavior of the flow is not only of interest within its own right but is also necessary if the computations for the quantitative establishment of the water surface profiles are to be carried out correctly. Various references considered the classification and computation of such water profiles. In order to obtain an expression from which the nature of the water surface profiles might easily be deduced, some assumptions are made by many hydraulic researchers. These assumptions can be summarized as follows:

1- The channel is wide.
2- The pressure distribution is hydrostatic,
3- The discharge per unit width is constant.
4- The velocity and pressure distribution factors are considered to equal unity.
5- The constant longitudinal bottom slope is small such that the depth measured vertically is approximately equal to the depth-measured perpendicular to the channel bottom.
6- Manning equation is used to eliminate the energy gradient, and express the resulting discharge in terms of the normal depth and the bottom slope.

With these assumptions and the relationship relating the flow discharge and the critical depth, the dynamic equation for the gradually varied flow is derived,

$$\frac{\partial E}{\partial x} = S_o - S_f \quad \text{……………………………………………….(2.3.1.1)}$$

Where;

$$E = h + {v^2}/{2g} \quad \text{………..…………………………………(2.3.1.2)}$$

E Represents the specific energy, h is the flow depth and v is flow velocity. The equation may be expressed also as;

$$\frac{\partial h}{\partial x} = \frac{S_o - S_f}{1 - Fr^2} \quad \text{…..……………………………………….(2.3.1.3)}$$

Where

$$S_f = (\frac{Qn}{AR^{2/3}})^2 \quad \text{………………………………………….(2.3.1.4)}$$

Given that S_o is the bed slope, S_f is the energy slope and Fr is the Froude number, Q is the flow discharge, n is Manning coefficient, A is the flow area and R is the hydraulic radius. The derived differential equation may further reduced to;

$$\frac{\partial h}{\partial x} = S_o \frac{1-\left(h_n/h\right)^{10/3}}{1-\left(h_c/h\right)^{3}} \qquad \text{.......................................(2.3.1.5)}$$

Where; h_n is normal flow depth, and h_c is the critical flow depth. The above equation represents the dynamic equation of the gradually varied flow.

To establish the water surface profile in either a natural or man made channel, one must take into account the losses due to boundary resistance and losses due to bends, expansions, etc. In essence, the procedure of establishing the non-uniform water surface profile consists of satisfying the energy equation. When the distance between the two sections of the reach and the depth at one section is known, the method used to solve the dynamic gradually varied flow for the unknown depth at the other section is called the standard step method. This is a trial and error method because the equation is an implicit expression in the unknown depth. In flow profile computation, beside the specified discharge, the water surface elevation at the control section and the geometric and roughness characteristics of the channel sections are generally required. Moreover, other methods such as the direct step method and the graphical-integration method are described many authors.

Generally, backwater curves of gradually varied flow cannot be established unless conditions are known at the starting section or can be established, or the computations started at a control section, where a unique relationship

between discharge and depth exists. Conceptually, there is nothing to prevent the standard step method from being used to carry back water calculations upstream or downstream whether the flow is sub-critical or supercritical. However, the numerical procedure is stable if the calculations are carried in the upstream direction in sub-critical flow and downstream in supercritical flow.

Many open channel flow phenomena of importance are unsteady in character. The flow equations in open channel, governing the most of the hydraulic cases, are the continuity flow equation and the equation of motion. Sent Vennat developed such equations and their solutions, which were investigated by many authors such as Strelkoff (1969), (1970) and Oosteveld and Admowski (1976). A lot of models are primarily devoted to compute the water surface profile. Chow (1959) and Henderson (1966), clearly, presented the different procedures for surface flow profiles computation. In addition, the solution characteristics of the flow profiles investigated by Pickard (1963), Vallentine (1964), Vallentine (1967), Rao and Sridharan (1966), Prasad (1970) and Lakshmana and Sridharan (1971).

Most of references employ the Manning equation in various ways to resolve the energy equation. Chow (1959) and Henderson (1966), among others, suggest the energy slope between to adjacent sections to be evaluated by;

$$S = \frac{S_{f1} + S_{f2}}{2} \qquad \text{..(2.3.1.6)}$$

In 1972, Morris and Wiggert took a different approach to evaluate the energy slope represented in that the average flow area and the average hydraulic radius are considered. In 1975, Venared and Street estimated the sections properties in Manning equation using the values in the definition of the energy slope;

$$S_f = \frac{Q^2 n^2}{\left(\frac{2A_1 A_2}{A_1 + A_2}\right)^2 \left(\frac{R_1 + R_2}{2}\right)^{4/3}} \quad \text{.................................(2.3.1.7)}$$

In computing an average reach roughness for natural waterway, some researchers used the geometric mean of the conveyances. Using this technique, the energy slope becomes;

$$S_f = \frac{Q^2 n^2}{\left(A_1^2 R_1^{4/3}\right)\left(A_2^2 R_2^{4/3}\right)} \quad \text{.......................................(2.3.1.8)}$$

Using the arithmetic mean of the conveyances, the energy slope may be written as

$$S_f = \frac{Q^2 n^2}{\left(\frac{A_1 R_1^{2/3} + A_2 R_2^{2/3}}{2}\right)^2} \quad \text{.......................................(2.3.1.9)}$$

Some authors proposed an averaging technique by using the harmonic mean of the upstream and downstream energy slopes. Accordingly, the energy slope for the reach becomes:

$$S_f = \frac{2S_{f1} S_{f2}}{S_{f1} + S_{f2}} \quad \text{...(2.3.1.10)}$$

2-3-2 VEN TE CHOW WORK

Chow (1959) stated clearly the similarity and differences between the applications of the energy and momentum principles on a rigid bed channel reach. In the energy equation, he mentioned, the head loss term measures the internal energy dissipated in the whole mass of the water in the reach, whereas the equivalent term in the momentum equation measures the losses due to external forces exerted on the water body by the walls of the channel. He considered that, if the small differences between the velocity distribution coefficients and the pressure distribution coefficients are ignored in gradually varied non-uniform flow, then it seems that the internal energy losses are practically identical with the losses due to external forces.

Some problems in hydraulics field such as hydraulic jump, as mentioned by Chow, if the energy equation is applied then the unknown internal energy loss being indeterminate. If instead of that, the momentum equation is applied, dealing mainly with the external forces, the effects of the internal forces will be entirely omitted and need not be evaluated. Generally, the energy principle offers a simpler and more direct solution and explanation than the momentum principle dose. In most of hydraulic engineering problems, the application of the energy principle is more in use.

2-3-3 FLOW OVER MOVABLE BOUNDARY CHANNELS

The behavior of a system with moving liquids and entrained solids is very complex as stated formerly, and depends on several factors; the mobility and geometric characteristics of the boundaries, the presence of an interface between two fluids of different density, the geometry of the solids and properties of the fluid. Most of the sediment transport simulation models involve a hydrodynamic module to evaluate the flow variables along the channel reach, and a morphological module to compute the bed changes in alluvial channels. Numerical simulations are usually carried, using various computer software codes to investigate natural river hydraulics. Most of these codes are based on the three dimensional Navier-Stockes equations for which detailed derivation can be found in fluid mechanics text-books such as Akode (2004). These equations can be written into Cartesian form as follows;

- Continuity equation:

$$\frac{\partial u_j}{\partial x_j} = 0 \qquad \text{………………………………………(2.3.3.1)}$$

- Momentum equations:

$$\frac{\partial u_i}{\partial t} + \frac{\partial (u_i u_j)}{\partial x_j} = F_i - \frac{1}{\rho}\frac{\partial P}{\partial x_i} + \frac{1}{\rho}\frac{\partial \tau_{ij}}{\partial x_j} \qquad \text{………………………….(2.3.3.2)}$$

Where F_i is external force per unit volume of fluid, ρ is fluid density, u_i $(i = 1,2,3)$ are the velocity components, P is pressure. τ_{ij} Are the turbulent stresses, z is bed level. The validity of these equations has been verified by many observations and experiments. The following paragraph concerned with the simple one-dimensional form of these equations, as it is necessary to investigate the assumptions and the basic considerations involved.

The law of continuity for unsteady flow may be established by considering the conservation of mass between two sections in a channel as presented by Chow (1959). In unsteady flow, the discharge changes with distance and the depth changes with time.

The change in discharge through space in the time dt is

$\left(\partial Q / \partial x \right) .dx.dt$.

The corresponding change in channel storage in space is

$\left(\partial A / \partial t \right) dx.dt$.

Since water is incompressible, the net change in discharge plus the change in storage should be zero; thus

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad \text{……………………………………….(2.3.3.3)}$$

This is the continuity equation for unsteady flow in open channel. The equation of motion for unsteady flowing water in open channel is conducted and derived in many references such as Chow (1959) and Henderson (1966). The variation in velocity of flow is taken into account and accordingly brings to the fore the acceleration or deceleration, which produce force and causes additional energy losses in the flow. The equation is given as follows;

$$\frac{\partial h}{\partial x}+\frac{v}{g}\frac{\partial v}{\partial z}+\frac{1}{g}\frac{\partial v}{\partial t}=S_o-S_f \quad \text{……………………………(2.3.3.4)}$$

Most of hydraulics textbooks define the shear stress developed on the boundaries of an open channel as the pull of water on the wetted area, known as the drag force or tractive force. Generally, it is believed that this concept has been first introduced in hydraulic literature by Du Boys in 1879. The shear stress τ is defined as follows;

$$\tau=\gamma RS_f \quad \text{…………………………………………..…(2.3.3.5)}$$

Where γ is the specific weight of the water. It worth mentioning that, shear stress is a significant parameter in alluvial channel modeling. Some of related works are those of Wilson (1966), Myers and Elsawy (1975), Whiting and Dietrich (1990) Paquier and Khodashenas (2002).

Free surface movable boundary flow implies that the moving fluid is transporting solid mater as in a sediment-laden stream. If the boundary of the channel is movable, then this type of flow has two additional degrees of freedom over open channel rigid boundary flow. The size, shape, density and gradation of the transported mater represent one degree of freedom, which influence the flow. The size, shape and position of bed forms, as contrasted with the individual particles can vary with time, this represents the other degree of freedom. As a consequence of these additional degrees of freedom, flow situations of this type are very complex. When water flows over a movable boundary channel, the bed surface is normally being deformed into various configurations. In such situation, the flow experiences a resisting force that opposing the motion. This resistance is called the drag. The total drag is composed of the skin friction and form drag. The skin drag, or friction, equals to the integral of all shear stresses taken over the surface of the channel boundary. Whereas, the form drag equals to the integral of all pressure taken over the surface of the channel bed in the direction of motion.

The shape, orientation and the surface roughness of the bed forms determine to a great extent which part of the total drag is due to form drag and which is due to skin friction. Since the skin friction and the form drag are governed by different sets of laws, it is reasonable to decompose the total resistance as proposed by several researchers before. The decomposition is as follows:

$$\tau A = \tau^{'} A + \tau^{''} A \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3.3.6)$$

Where τ is the total shear stress on the bed, A_b is the area of the bed, $\tau^{'}$ is the skin shear stress on the bed and $\tau^{''}$ is the form shear stress on the bed. In a steady uniform flow and assuming that the total shear characteristics does not vary in its transverse direction.

$$\gamma RS_f = \gamma R^{'} S_f + \gamma R^{''} S_f \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots..(2.3.3.7)$$

The energy slope can also be divided;

$$\gamma RS_f = \gamma RS_f^{'} + \gamma RS_f^{''} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3.3.8)$$

The interaction between the flow of the water-sediment mixture and the riverbed creates different bed configurations, which change the resistance to flow and rate of sediment transport. The gross measures of channel flow such as flow depth, bed elevation and flow velocity change with different bed configurations. In a natural stream it is possible to experience a large increase in discharge with little or no change in stage as a result of change in bed form. Conversely, change in flow depth with constant slope and bed material can change the bed forms.

If the sediment and the water discharge are the primary independent variables influencing the channel morphology, then it would be possible to show quantitative relations between water discharge, the nature and quantity of the sediment, and all aspects of channel morphology such as channel dimensions, shape, gradient and pattern. In 1955, Lane concluded that a channel could be maintained in a dynamic equilibrium by balancing changes in sediment load and sediment size by compensating changes in water discharge and channel gradient. Exener, in 1925, proposed the equation of sediment continuity to be written in the same manner of the water continuity equation;

$$\frac{\partial z}{\partial t} + \frac{1}{(1-p)} \frac{\partial q_{sj}}{\partial x_j} = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3.3.9)$$

q_{sj} represents the sediment transport rate per unit width and p is the porosity.

2-3-4 MIXED AND ARMORED LAYERS

In most sediment transport studies, the treatment of the bed material layer is considered as an important parameter that plays effective role in sediment transport modeling. Einstein assumed, in alluvial channels, the thickness of the bottom layer, in which the movement of the bottom layer takes place, equals twice the representative particle diameter. Henderson (1966) suggested the active bed material layer thickness, E_m, to be given as follows;

$$E_m = 11 S_f h \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.3.4.1)$$

In which S_f is friction energy slope and h is the water depth.

Karim and Kennedy (1982) defined the horizon of bed material undergoing continual

mixing due to turbulence, bed form-migration, etc. as the mixed layer and is assumed to be homogeneous in its size distribution at any time. The thickness of mixed layer is assumed to be equal to the average height of the bed forms. When the bed degrades, sediment with the parent bed material size distribution enters the mixed layer from below, in case of an aggrading bed, material with the size distribution of the mixed layer leaves the mixed layer and becomes part of the inactive layer. The quantity of material entering or leaving the mixed layer during a time-step depends on the amount of degradation or aggradation, and the thickness of the mixed layer in the previous and current time steps.

Elsadig M. Abdalla et al (1986) developed a method to estimate the bed material transport, as continuous function of initial and boundary bed conditions, by introducing a time-space dependent coefficient, which is linked to grain availability and varying bed composition. That coefficient is the bed surface composition and is introduced as a supplementary state to describe the bed condition for the sediment transport. A schematic representation of a channel reach was given to define the different solid material granulometric distribution for sediment mixture.

Rahuel (1989) mentioned that the interpretation of the mixed layer could not be disassociated from the time scale under consideration. If a very short time scale is considered, then the mixed layer can be considered as a thin surface layer containing particles that susceptible to entrainment into the flow due to increase in the local bed shear stress. If the considered time scale is somewhat larger, the mixed layer can be thought of as occupying the vertical space traversed by the bed forms in their downstream movement.

Borh (1989) mentioned that the larger and heavier particles, which are non-transportable under the flow conditions, remain on the bed and gradually occupy the entire bed surface forming what is known as the armor layer. In 1991, Kulkarni continued to state that, after the formation of the armor layer, the finer particles in the bed material escape through the openings in the armor layer, though at much slower rate, till a filter is naturally developed under the armor layer. The armor layer gradation is characterized by its median diameter since it contains particles greater than the smallest non-transported particle, and also smaller ones are hidden by the larger particles.

Niekerk et al (1992) mentioned that the bed region is divided into three horizons:

- An upper zone, termed the mixing layer, representing the space occupied by bed form.
- A top horizon of the mixing layer, termed an active layer, in which continuous exchange of sediment particles between the bed and flow takes place.
- The subjacent static bed.

Cui et al (1996) proposed a three-layer model for the treatment of sediment conservation. The model includes the bed load layer, the active layer and the substrate layer. In case of degradation, it is assumed that the active layer mines the substrate layer as the bed degrades. While in the case of aggradation, it is suggested that the active layer acts as a kind of filter, such that bed load is transferred to the substrate layer as bed aggrades. Singh et al (2004) proposed the following expression for estimation of the thickness of the active layer;

$$E_m = d_{90} + 0.3h\left[1 - \frac{\tau_c}{\tau}\right] \qquad \text{……………………………..….(2.3.4.2)}$$

Here d_{90} denotes the sediment size 90% finer of bed material.

2-4 MECHANICS OF SEDIMENT TRANSPORT

Sediment transport involves a complex interaction between numerous interrelated variables, as mentioned before. However, theoretical approaches in sediment studies are based upon simplified and idealized assumptions. It has been common practice to assume that the rate of sediment transport or the magnitude of sediment concentration can largely be determined by certain dominant variables such as water discharge, velocity, the energy gradient, shear stress, stream power, unit stream power, relative roughness, the Froude number, etc. The large number of methods that have been developed to estimate the transport rates is normal to be expected for the significance of the problem of sediment transport. The prediction equations for computing sediment transport rates have some input variables in common. The input requirements of such predictors belong to one of three classes: flow parameters, soil particle parameter and channel characteristics.

2-4-1 PROPERTIES OF SEDIMENT

Sediment may be composed of different kinds of particles varying in size, gradation and specific weight. The size of sediments, the fall velocity of a single particle or of a group of particles, and the specific weight of a single particle and the characteristics of deposited sediment are considered to be very important properties. Their importance refers to their use to assess the life of reservoir, the evaluation of the scour depth, the siltation rate in estuaries, the dimension of scale physical models, etc. To estimate the resistance to flow or the rate of sediment transport, these properties or some of them have to be fully understood in order to thoroughly investigate the problems related to sediment transport.

The physical properties of the individual particle, such as the particle size, shape, density, specific weight and fall velocity and the bulk properties of the sediment are very significant in studying the mechanics of sediment transport in alluvial systems. The size of the sediment particles is of a great significance, not only because size is important and the most readily measured property, but also because other properties such as shape and specific weight tend to vary with particle size. Moreover, the size distribution of the sediment that forms the bed and banks of the channel are of great significance and so the distribution of the sediment particles entering a channel reach.

The shape of the sediment particles may vary geometrically but the most pertinent parameter is the sphericity, which describes the relative motion between the falling particle and the fluid. On the other hand, the roundness has a small effect on the hydrodynamics behavior of the particles. Other properties such as the density, which is a function of the mineral composition of the sediment particles, and the specific weight are important factors extensively used in hydraulics and sediment transport.

Another primary parameter defining the interaction of sediment transport with the bed or banks is the fall velocity of sediment particles. It has been shown that the bed configuration in a sand channel may change when the fall velocity of the bed material changes.

The most important bulk properties of sediment are the size distribution, specific weight and the porosity of the bed material. Richardson describes methods of analyzing the size distribution of different classes of sediment materials, in 1971. Nevertheless, it is necessary to make statistical analysis of particles size in order to define fully the representative particle size of any sediment mixture as a whole. One of the important variables in many problems is the angle of repose of sediment. It has been incorporated in many sediment discharge predictors. The angle of repose of sediment particles can be determined by introducing particles of sediment into nearly static water and then measure the critical toe angle of the sub-measured cone of the deposited sediment.

Most natural soils have a certain amount of cohesion, which is a property mainly defined by empirical relations. When the effect of cohesion is negligible, the loose material covering the bottom of streams can be treated theoretically. Generally, sediments are broadly classified as cohesive and non-cohesive. With cohesive sediment, the resistance to erosion depends on the strength of the cohesive bond binding the particles. Once erosion has taken place, cohesive material may become non-cohesive with respect to transport. On the other hand, the non-cohesive sediments generally consist of larger discrete particles than the cohesive soils. Non-cohesive sediment particles react to fluid forces and their movement is affected by the physical properties of the particles. Simons and Senturk (1992) mentioned that Rouse, in 1950, presented table (2.4.1) for sediment grade scale.

Table (2.4.1) Sediments Grade Scale

Size in mm	**Inches**	U.S. Standard	*Class*
4000-2000	160-80	-	*Very large boulders*
2000-1000	80-40	-	Large boulders
1000-500	40-20	-	Medium boulders
500-250	20-10	-	Small boulders
250-130	10-5	-	Large cobbles
130-64	5-2.5	-	Small cobbles
64-32	2.5-1.3	-	Very coarse gravel
32-16	1.3-0.6	-	Coarse gravel
16-8	0.6-0.3	2.5	Medium gravel
8-4	0.3-0.15	5	Fine gravel
4-2	0.16-.08	9	Very fine gravel
2-1	-	16	Very coarse sand
1-1/2	-	32	Coarse sand
½-1/4	-	60	Medium sand
¼-1/8	-	125	Fine sand
1/8-1/16	-	250	Very fine sand
1/16-1/32	-	-	Coarse silt
1/32-1/64	-	-	Medium silt
1/64-1/128	-	-	Fine silt
1/128-1/256	-	-	Very fine silt
1/256-1/512	-	-	Coarse clay
1/512-1/1024	-	-	Medium clay

1/1024-1/2048	-	-	Fine clay
1/2048-1/4096	-		Very fine clay

(Source: Simons and Senturk (1992)

2-4-2 INITIATION OF SEDIMENT TRANSPORT

Water flowing over a bed of sediment exerts forces on the grains. These forces tend to move or entrain them when they reach the critical or threshold conditions. It is possible to assess the threshold condition for the beginning of particle motion, which depends on channel geometry, flow conditions and sediment characteristics. The ratio of the drag force to the gravitational force, which is a dimensionless parameter, a type of Froude number, is an important parameter that is related to the grain size and the shear velocity. Considering the lift force Aksoy, in1973, found that, for Reynolds number ranging from 2700 to 6600, the lift force fluctuated around 1/7 of the drag force. Coleman's experiments, in 1972, showed this ratio was about unity. For completely developed turbulent flow, the shear stress is proportional to the velocity near the bed. The stage of the phenomenon corresponding to the initiation of sediment transport is referred to as the critical stage. In accordance with practice, it is assumed that the fluid and granular material are specified and the phenomenon varies with the flow parameters, the shear velocity and the flow depth. The critical stage is given by a certain condition that satisfied by the dimension less variables of the flow. In general, the beginning of motion is difficult to define. This difficulty is a consequence of a phenomenon, which is random in space and time.

Kramer, in 1935, has defined the types of motion of bed material as weak movement, in which only a few particles are in motion on the bed, medium movement, in which the grains of mean diameter begin to move, and general movement, in which all the mixtures is in motion. In reality, there is no truly critical condition for initiation of motion for which motion begins as the condition is reached. Research conducted on the initiation of particle motion has almost utilized nearly uniform material. Many researchers such as shields, in 1936, Simons and Richardson (1961), Vanoni (1974), etc, have attempted to solve the problem of the initiation of motion. For application to the motion of non-uniform granular material, the median grain size is suggested to represent the sediment mixture. The problem addressed for first time in terms of similitude by Shields. The graphic representation of this relation, known as shields diagram, is possibly one of the most frequently cited relations in the field of sediment transport and is certainly the most widely accepted criterion for the determination of the beginning of sediment transport. Shields used the overall bed shear stress without differentiation between form drag and surface drag. This resulted in critical values of shear up to 10 percent higher than for incipient motion on a flat bed. This error corrected, in 1971 by Gessler.

The pioneering work of Shields described the initial movement of uniform sediments on a planer bed under a unidirectional stream flow. Although his diagram is widely used (Task committee, 1966) expressed considerable dissatisfactions. However, Shields diagram has been later refined and modified by Yalin and Karahan (1979). Neill and Yalin (1969) described quantitatively the beginning of the sediment transport. Many researchers also developed models for sediment threshold on planer beds. For sediment threshold on an arbitrary sloping bed, a vector equation has been developed by Dey (2003).

Pilotti and Dilar (2001) distinguished between the beginning of sediment motion and the beginning of sediment transport. The former results as deprived of meaning, in consideration of the random nature of the grain entrainment process. The latter instead retains a conventional value that requires the explanation of the criterion used in its definition. They also mentioned that it is important to emphasize the strong stochastic component of the process in the field of sediment transport. Dey (2003) presented theoretical and experimental investigations on the threshold of non-cohesive sediment motion under a steady-uniform stream flow on a combined transverse and longitudinal sloping beds. Theoretical analysis of the equilibrium of a sediment particle showed that the critical shear stress ratio is a function of the transverse slope, longitudinal slope, angle of repose and the Lift-drag ratio. The theoretical model agreed closely with the experimental results. Papanicolaou et al (2004) illustrated the inappropriateness of the traditional Shields criterion for use in mountains streams due to the lack of plane bed configurations and sediment size uniformity.

2-4-3 SEDIMENT MOVEMENTS

Mc Cuen (1989) divided the sediment load transported by a channel into bed-material load and wash load, defining the earlier portion as that one composed of grain sizes originating in the channel bed and sides while the wash load is composed of finer-drained particles with virtually no settling velocity and which originate from the land surface of the watershed. The bed-material load is further divided into bed-load and suspended load. Generally, the bed-material particles are transported by flow in one or more combination of ways:

1. Rolling or Sliding on the bed.
2. Jumping into the flow and then resting on the bed, which is known as Saltation.

3. Supported by the surrounding fluid during a significant part of its motion, which is known as suspension.

There is no clear demarcation to differentiate between the ranges of sediment sizes and how they move. In general particles vary in the degree in which they are suspended in the flow and in which they roll or jump along the bottom. Most of researchers differentiated types of sediment movement into two types. Sediments, which are suspended in the flow, and sediment transported by a flow in the form of bed load, which depends upon the size of the bed material particles and the flow conditions.

2-4-3-1 BED-LOAD

The bed-load is the material that too coarse to be supported in the flowing water. Although the amount of bed load may be small, as compared with the total sediment load, it is very significant because it shapes the bed and influences the stability of the channel, the grain roughness and the form of bed roughness. The present work is mainly concentrates on the solution of the sediment transport problem in alluvial channel considering the bed-load sediment as a deriving parameter.

Computation of bed load rate is one of the main research sectors in sediment transport mechanics therefore, the relations used in computing bed load are considered later in this work. Appendix (B) presents some details of bed-load transport.

2-4-3-2 SUSPENDED LOAD

The suspended load can be classified as all the particles that are lifted up by eddies in the flow and move long distances down stream before settling to the bed. The fluid continuously supports the weight of suspended sediment particles. Turbulence is the most important factor in the suspension of sediment. Owing to the weight of the particles, settling is counter-balanced by the irregular motion of the fluid particles introduced by the turbulent velocity components. Suspended Sediment in a stream channel has a vertical distribution, less dense near the water surface and more dense near the bed. This distribution is determined by the balance between the rate at which particles are falling due to gravity and the rate of moving up again by turbulent motion.

2-4-3-3 TOTAL LOAD

Total load is the sum of the bed load and suspended load, or the sum of the bed material load and wash load. Simons and senturk (1992) stated, in research work, it is normally dealt with bed material load and wash load in uniform flow separately, because the wash load is determined by available upslope supply rate and can not be predicted by the transport capacity of the stream.

2-4-4 SEDIMENT MIXTURES

The determination of the critical condition for the sediment incipient motion and sediment transport rate is very important. The particles more prone to move first, those have higher drag and less weight. For study of incipient motion, sediment particles are divided into groupings related to the uniformity of the particle size distribution. After the work of Du Boys, in 1879, on bed load transport and the curve proposed by Shields for the prediction of the critical shear stress of incipient motion, the uniform sediment movement has been extensively investigated and the transport mechanism is well understood. However, estimation of the non-uniform sediment transport is still inadequate.

Much of the development in the analysis of the bed load of uniform sediment was influenced by the work of Du Boys, in which it was assumed that the bed material moves in layers and the difference in mean velocity of the successive layers increases linearly towards the bed surface. The critical shear stress affect the equilibrium condition of the particles is determined through Shields Diagram. In addition to the above-mentioned work, various research works have been carried considering the transport of uniform bed material.

There are some of the pioneering researches to fractionally calculate the non-uniform bed load transport rate. These mentioned by many authors such as Misri (1984), Chiew (1991), Patel and Raju (1996). Proffit and Sutherland, in 1983, proposed and exposure correction factor for the Ackers and White’s (1973) bed load transport formula. Garbrech et al, in 1995, used three different established transport relations to calculate the transport rate for different size classes. The transport relations are: Laursen’s formula for size classes from 0.01mm to 0.25mm, Yang’s formula for size classes from 0.25mm to 2.00mm and Meyer Peter and Muller’s formula for size classes from 2.00mm to 5.00mm. Then the total sediment discharge is calculated.

Wu et al (2000) proposed a formula for fractional bed load transport capacity

$$\phi_{bk} = 0.0053\left[\left(\frac{n'}{n}\right)\frac{\tau_b}{\tau_{ck}} - 1\right]^{2.2} \qquad \text{……………………………(2.4.4.1)}$$

Where ϕ_{bk} is non-dimensional bed load transport capacity, n is Manning roughness coefficient for channel bed, n' is Manning grain roughness, τ_b is bed shear stress and τ_{ck} is the critical shear stress for the k-th size class.

For fractional suspended load transport capacity;

$$\phi_{sk} = 0.0000262\left[\left(\frac{U}{\omega_{sk}}\right)\frac{\tau}{\tau_{ck}} - 1\right]^{1.74} \qquad \text{…………………………(2.4.4.2)}$$

Where ϕ_{sk} is non-dimensional suspended load transport capacity, τ is shear stress of entire section, U is the flow velocity ω_{sk} is settling velocity for the k-th size class.

It worth mentioning that, sediment material consists of a mixture of clay-sized particles and sometimes of sand-sized particles. Cohesive soil properties depend upon mineral composition and the interaction between the particles and water. The properties also depend on the state and history of consolidation. In addition, the flocculation is more intensively to develop in cohesive sediment. The flocs of sediment particles behave much differently from the individual particles. Different charts and diagrams have been proposed to show the typical trend of the settling velocity of the flocs when the sediment concentration increases. Researches carried on cohesive sediment are so many such as Partheniades (1965), Nicholson and O'conner (1986), Mehta (1989), etc.

Deposition rate of the cohesive sediment is determined by several methods. Most of them depends on the bed shear stress which compared with the minimum critical bed shear stress below which all sediment are deposited on the bed; and the maximum critical bed shear stress above which all sediment remain in suspension yielding a zero deposition rate. On the other hand, the erosion rate of the cohesive sediment is determined by a method relating the sediment properties, such as mineral composition, organic material, salinity, etc. to the critical shear stress for erosion which depends on dry density, temperature, etc. The consolidation of bed material decreases the bed elevation. This compaction process of the deposited material under the influence of the gravity forces with a simultaneous expulsion of pore water a gain in strength of the bed material depends on the dry density. The dry density for consolidated sediment is determined as a function of time.

2-4-5 SEDIMENT LOAD COMPUTATIONS

The tractive-force formula of Du Boys, in 1879, was introduced as the first work on sediment transport computation. Bogardi (1974) mentioned that, almost hundred years before Du Boys, a theoretical expression derived by P. Du Buat for the frictional force developed between the channel bottom and the column of water moving above it. Following the assumption of Du Buat, the friction developed on the channel bottom was offered by Du Boys as an explanation for the movement of the sediment particles. The concept of the tractive-force is thus closely associated with the movement of bed load.

Du Boys interpretated the magnitude of the tractive-force equals the friction developed on the channel bottom. He assumed that an increase in the kinetic energy of flow to be offset exclusively by the work performed by friction on the channel bottom, i.e. by the tractive-force. Bogardi (1974) stated that the

other resistances, internal friction, vortices air resistance, etc, to be overcome by conversion of potential energy into its kinetic counterpart. Thus, as he mentioned, the most serious objection to the Du Boys formula that, only part of the kinetic energy consumed by the friction force developed on the channel bottom.

Beside the tractive force, the mean velocity of the flow is used for describing the incipient condition of sediment transport. The classical theory in which the critical condition is described is referred to as the impact theory. Numerous formulae for estimating the critical tractive force and the critical velocity have been developed by several researchers. A number of expressions for critical tractive force derived, on the basis of laboratory experiments, have been published in the literature. In 1936, experiments performed by Shields for determining the critical tractive force. The results of these experiments are shown by graphical representation. In 1937 Bogardi and

Yen have shown that the magnitude of the critical shear to depend also on roughness condition. Other expressions suggested by Schoklisch, Straub and Chang. Kramer, in 1938, suggested a formula based on constant tractive force. From experiments carried out at the U.S. Waterways Experiments Station, Kramer formula was modified. Lane recommended curves of critical tractive forces for use in connection with irrigation canals.

In1942, Kalinske developed a bed load equation based on considerations given to turbulent fluctations. Meyer-Peter and Muller, in 1948, developed an equation based on experimental work with sand particles of uniform size. The equation based on the portion of the total bed shear that is effective in moving the bed particles. Starting from the relation of Shields, Egiazaroff (1965) derived an expression in which he introduced a dimensionless resistance coefficient that depends on the fall velocity. He assumed that at a particular

velocity distribution in the critical condition, the velocity to which the particles is subjected may be taken identical with the fall velocity of sediment particles of comparable size and specific gravity in clear water at rest.

Instead of dealing with the concept of the tractive force, Einstein, in 1937 and 1950, considered the dependence of the probabilities of movement or deposition of sediment on flow characteristics. He developed the first stochastic model to describe the motion of a sediment particle that moves with the bed in a series of alternating transport and rest periods. He assumed that during the transport periods a particle may roll along the bed, jump as saltation or may be suspended by the flow. Einstein assumptions were as follows:

1- The velocity field is stationary in time and homogeneous in the lateral as well as in the longitudinal directions.
2- The transport periods of the sediment particles are in significantly small as compared to the rest periods.
3- The probability for a particle to be moved by flow is independent of its location.

Polya reviewed Einstein's development and developed a partial differential equation based on a simple model. He assumed that the rate of change of the bed load discharge by weight at a certain section is equal to the difference between the accumulation rate and the depletion rate. He further assumed that the accumulation rate is proportional to the bed load flux passing the section while the depletion rate is proportional to the existing bed load concentration.

Bagnold, in 1966, stated that the rate of work done is a product of the available stream power and the efficiency. He further mentioned that the bed load work rate is a product of the bed load transport rate and the coefficient of dynamic solid friction. For graded sediments, he proposed to use a representative size. Engelund and Hansen used Bagnold's stream power concept and the similarity principle to obtain another sediment transport equation.

De Vries, in 1967, developed a diffusion model for the dispersion of the bed material particles. He started with the equations of the continuity and motion to develop his diffusion equation using the following assumption:

1- The transport condition is constant in time and space.
2- The variations perpendicular to the main current will be neglected.
3- The bed material is uniform.

The concept of stream power is also strongly used in sediment transport studies by Yang (1972, 1976), Chang and Hill (1977), and Yang and Stall (1976). Descriptions of sediment transport theories and equations found in a variety of references such as those by Graf (1971), Yang (1973), Bogardi (1974), Karim and Kennedy (1982), Chang (1984) and Hussein (1994).

A rich source of literature on the sediment transport rate predictors is Simons and Senturk (1992), which presents thoroughly all pervious work of most of researchers. Chanson (1999) presented various Empirical and semi-empirical correlations of bed load Transport. A lot of works have been published concerning the measurement of sediment transport rate and computation. Such investigations include Colby (1964), Bishop (1965), Grag (1971), Einstein and Aal (1972), Kikkawa and Ishakawa (1978), Van Rijn (1984, 1986), Samaga et al (1986), Low (1989), Nakato (1990), Leo et al (1991), {E24 – E28}, etc. Papanicolaou (2004) mentioned to the main recent researches such as Lopes et al, 2001, which evaluated selected bed load equations. Appendix (B) represents part of bed-load computations. Table (2.4.2) highlights on some sediment transport formulae.

Table (2.4.2) Some Sediment Transport Formulae

Author(s)	Input Parameters	Type	Remarks
Velikanov, 1954	τu	G.	Suspended Load
Laursen, 1958	u_*, w_*	C.S.	Total Load
Brooks, 1963	u, u_*, C_s, q	S.V.	Suspended Load
Colby, 1964	u, h, d_s	S.P.	Total Load
Bishop et al, 1965	ϕ_T, ψ'	Pr.	Total Load
Bagnold, 1966	$\tau u, e_b$	S.P.	Bed Load
Blench, 1966	q, S, d_s	R.	Total Load
Engleund&Hansen, 1967	τ, τ_c, K	C.S.	Total Load
Chang et al, 1967	h, u_*, C_s	S.V.	Suspended Load
Toffaletti, 1969	ϕ_*, ψ_*	Pr.	Bed Load
Shen& Hung, 1971	u, S, w	Reg.	Total Load

Ackers& White, 1973	h, u, S, d_s	Reg.	Total Load
Karim& Kennedy, 1981	q, S, d_s	Reg.	Total Load
Van Rijin, 1984	h, u, d_s	Reg.	Suspended Load
Simons& Senturk, 1992	q, q_c, S, K	C.Q.	Bed Load
Singh, 2004	τ, τ_c, q, K	C.S.	Bed Load

C.S.: Critical Shear, C.Q.: Critical Discharge, C.V.: Critical Velocity, Pr.: Probability, S.V.: Shear Velocity, G.: Gravitational Theory, S.P.: Stream Power, R.: Regime Theory, Reg.: Regression

2-5 NUMERICAL MODELING OF ALLUVIAL CHANNELS

Alluvial rivers are increasingly exploited for the beneficial uses such as water storage in reservoirs, hydropower generation, bed material mining, etc. These kinds of beneficial uses can destroy the natural equilibrium of the river and so doing morphological changes in the river environment. These often require costly compensating engineering measures to stabilize the river-bed. Therefore, some means of predicting the medium and long-term effect on bed equilibrium are in need. Reduced-scale physical modeling is entirely appropriate when local problems are under study. On the other hand, physical modeling is not generally feasible when large spatial extents and long time periods are to be studied. This is often the case in river engineering. Numerical modeling is the suitable means of studying bed evolution in such cases.

Mathematical modeling, defined as a symbolic representation of a situation using a set of mathematical equations, is commonly used in hydraulic engineering problems. Effective modeling requires accurate theoretical basis that describes the physical phenomenon to be modeled and accurate field or laboratory data to calibrate and validate the model. In general, the selection of a model depends on the type and scale of the problem considered, the availability of the input and calibration data, the degree of physical schematization, the specified accuracy and the available budget.

Existing mathematical models are nearly based upon the idea that it should be possible to simulate hydrological flow conditions and the changes in longitudinal profile of a river over a period of 2 – 50 years. In mobile-bed river hydraulics, most of the mathematical models represent longitudinal bed profiles; longitudinal free surface profiles and sediment transport as a function of time and hydraulic flow conditions. Nevertheless, such models can be used to solve numerous problems associated with riverbed evolution in response either to natural conditions or man-made developments. Some of the natural phenomena that can be successfully simulated with such models are:

- Delta formation in reservoirs and at river mouths.
- Bed variation downstream of tributaries or at bifurcations.
- Long-term natural evolution of a river-bed.

There are two general classes of equations appearing in mobile-bed modeling. The first class comprises conservation equations, generally in the form of linear and nonlinear partial differential equations. The second class comprises semi-empirical relations representing mathematical formulations of poorly understood complex physical processes of sediment transport rate and roughness friction factor, may be algebraic or differential equations. The essential feature of a water-sediment routing model is the solution of these five, strongly coupled relations:

1- The equation of continuity of flow.
2- The equation of motion of flow.
3- The equation of sediment continuity.
4- The relation of roughness of sediment-transporting stream.
5- The relation of sediment discharge.

A complete mathematical description of river processes requires the solution of these governing equations, which include both time and space derivatives. Such a solution requires a prohibitive amount of computation time, and may not even be justified in view of the uncertainty in the formulations of some aspects of the physical processes.

2-5-1 MODELS DISCRETIZATION

In any model, it cannot be expected to obtain useful results unless physical reality by the model elements is correctly presented. The need for correct representation is related to two levels of the model formulation process; the hydraulic and topographic discretization. This refers the detailed hydraulic and topographic descriptions of flow cross-sections, flood plain cells, etc. as well as the choice of appropriate hydraulic equations. Modeling process requires that a series of computational points be selected along the watercourse and the flow equations, represent hydraulic laws, to be related to the flow variables from one point to another. There are several ways of describing the variation in the cross sectional area with the bottom elevation. It can be assumed that the cross section rises or falls with out changing its shape. In some models, an attempt is made to introduce a lateral distribution of deposits or erosion related to the shear stress.

Basically, models of alluvial channels require less precision than flood propagation models; since it does not usually known how to disturb the deposited or eroded material laterally. It may be a miss leading to represent cross sectional features in a sophisticated way. It is often preferred to use a rectangular cross section as representative of the real section. A very sophisticated representation of the deposits within a section may be inconsistent with other elements of the model (Cunge et al, 1980).

2-5-2 SIMPLIFICATION OF MODEL EQUATIONS

Many of mobile-bed modeling systems, as mentioned before, are based upon conservation of water mass, water motion and sediment continuity. The problem can be considered as a two-phase: liquid and solid. Under the assumption that all functions are continuous and differentiable, the working partial-differential equations are derived. The classical de Saint-Venant one-dimensional equations for the liquid phase are usually used in computation of water surface profiles. The bed load movement is conceptualized as occurring within a shallow region at the bed surface. Sediment discharge is computed at each section and erosion and deposition in each sub-reach is calculated by applying sediment continuity equation between the two bounding sections.

The governing equations in mobile-bed modeling system can be summarized in a simplified form as follows:

- The equation of continuity of flow, as stated before; for a rectangular channel of infinite width, the previous equation can be written as follows;

$$\frac{\partial q}{\partial x}+\frac{\partial h}{\partial t}=0 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.5.2.1)$$

- The equation of motion of flow can be written as follows;

$$\frac{\partial}{\partial t}\left(\frac{Q}{A}\right)+\frac{\partial}{\partial x}\left(\frac{Q^2}{2A^2}\right)+g\frac{\partial y}{\partial x}+gS_f=0 \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.5.2.2)$$

- The equation of sediment continuity;

$$\frac{\partial z}{\partial t}+\frac{1}{(1-p)}\frac{\partial q_s}{\partial x}=0 \qquad \text{……....……………………………(2.5.2.3)}$$

Where, q is the discharge per unit width, $y=h+z$ is the water surface elevation. The set of the above governing equation can be written in the following compact form:

$$\frac{\partial f}{\partial t}+\frac{\partial r}{\partial x}+k=0 \qquad \text{………………………………………..(2.5.2.4)}$$

Where

$$f=\begin{pmatrix} A \\ Q/A \\ z \end{pmatrix},\ r=\begin{pmatrix} Q \\ Q^2/2A^2+gy \\ \frac{1}{(1-p)B}G_s \end{pmatrix},\ k=\begin{pmatrix} 0 \\ gS_f \\ 0 \end{pmatrix} \qquad \text{…………(2.5.2.5a,b,c)}$$

Equation (2.5.2.4) represents a system of hyperbolic partial differential equations. Many analyses employ the quasi-steady approximation of De Vries, in 1967, according to which the flow equations are taken as steady when the characteristic wave of bed perturbations is small compared to that of water surface perturbations. This procedure results in a considerable numerical simplification in that the flow equations can be decoupled in time from the sediment continuity equation.

In the decoupled formulation, for a computational time step Δt, first the parameters of the equations related to the liquid phase flow are solved along the watercourse. It is assumed that one dependent variable can be computed independently from the other dependent variables during the time step. The solution consists of water depths, discharges and velocities computed at the initial time at all computational grid points of the model. The water depths and velocities found from the first step are used in the sediment transport formula and then the partial differential equation describing propagation of the bottom sediment wave is solved numerically. On the hand, the coupled formulation solves simultaneously the whole system of algebraic equations, the water phase and the sediment phase.

Generally, Numerical models of sediment transport need upstream and down stream boundary conditions. Internal boundary conditions are usually needed at all sections where the governing equations are not valid. External boundary conditions are applied to the limits of model. In some cases, the model may have several such limits, as the case of a network of channels, where external boundary conditions are to be imposed.

2-5-3 TYPES OF NUMERICAL MODELS

As far as the number of dimensions in space is concerned, water-sediment routing models are classified into one-dimensional, two-dimensional horizontal, two-dimensional vertical and three-dimensional models Siyam and Akode (2001). The general structure of these models starts with the analysis of the existing data, such as flow patterns, bed material composition, nature of sediment, development of bed contours …etc. For uncoupled solution scheme, the second stage, the hydrodynamic model, concerning with water continuity and water momentum equations, is applied. The results of the hydrodynamic model are used for the morphological model, in which the sediment transport rates, hence the bed changes, are computed

In one-dimensional models, only the cross-sectional average value of parameters are considered and bed changes can be predicted. Those models currently constitute the most widely used sediment transport models to simulate long-scale morphological changes. In general, there is a large need for experimental studies to guide the morphological research and calibrate the one-dimensional numerical models. The morphological changes in rivers usually regard long river-reaches and large durations are involved. This makes the possibility of calibration with prototype data very restricted. De Vreis (1994) focused on unsolved problems in one-dimensional morphological models. The assumptions upon which these models are based are in need to be investigated from the basic principles of fluid and sediment transport mechanics

Based on the objectives of the study, available data, computational resources, accuracy requirement and real time operational efficiency, and both the quasi two dimensional, one dimensional, or fully two-dimensional modeling approaches may be used. The typical data requirement to setup one-dimensional model, can be divided into two categories that are the boundary conditions and the topographic data. Series of discharges and water levels at upstream and downstream model boundaries are required to satisfy the boundary conditions of the model. Cross sections of the river are necessary to define topographic setup of the model. In one-dimensional modeling approach, simplified equations of continuity and momentum allow the use of large spatial resolution, thus making solution scheme more efficient.

Two-dimensional models provide predictions of more adequate accuracy. Most of the existing two-dimensional models are obtained by depth averaging leading to the two dimensional horizontal models. For cases in which flow variations are important over the depth, the most appropriate models are the two dimensional vertical models which are derived by integrating across the width to arrive at the laterally averaged equations of motion. Laterally averaged two-dimensional models are appropriate for modeling processes, such as density currents, thermally stratified flows and other flows in long relatively narrow reservoirs where the water surface level does not vary significantly, and there are no lateral inflows and outflows. The advantage of using the two dimensional approach is that it provides information for variable velocities and depths at any point of interest in the model domain. The computation of velocity profiles in two dimensions provides a better prediction of the effects of scouring and sediment transport processes.

Three-dimensional models are also of wide use in Mobile-bed Rivers and reservoir studies. In such models, the continuity and momentum equations in all directions are considered. Currently, three-dimensional models are powerful in showing the flow patterns such as the effect of secondary currents. However, as far as morphological processes are concerned, these models are only used to predict the initial rates of sedimentation and erosion to provide good insight into the short-term processes. The long-term morphological evolution requires a high computer power and space. In this respect, both the two-dimensional and three-dimensional models are heavily dependent on the parameters and requirements of computer space and time that limits their application.

2-5-4 COMPUTATIONAL TECHNIQUES

As mentioned before, numerical models consist of number of governing relations, often have different domains of validity. Moreover, there may be several methods of numerical solution of each of the basic sets of the flow relationships. Such numerical methods approximate the conservation relations and have some implications and limitations on numerical models. Convergence and stability analysis should be concerned when a numerical method is applied. Any proposed numerical method must be judged not only on the basis of its behavior when applied to a simplified system of linear equations, but also on the way it treats internal and external boundary conditions. Many sources in the literature mentioned to calibration of models and to the stability of their numerical computation, such as Fread and Smith (1978), Lyn and Goodwin (1987), Tinsanchali et al (1989), etc

In modeling alluvial channels, there are mainly three classes of numerical solution methods can be found in literature. Namely, these are:

- The method of characteristics.
- The finite element method.
- The finite difference method.

2-5-4-1 METHOD OF CHARACTERISTICS

The method of characteristics is used primarily in exceptional cases; nevertheless, because of the physical significance of its parameters and its capability to follow individual perturbations. The approach considered in this method is a semi graphical by which numerical solutions can be worked out. There is some literature on the application of the process in mobile-bed modeling as those of Cunge et al (1980), Lin and Shen (1984).

2-5-4-2 FINITE ELEMENT METHOD

The fundamental concept of the finite element method is the idealization of the actual prototype channel. Considering one-dimensional models, as an assemblage of finite number of individual elements, sub-reaches are interconnected at a finite number of nodal points, sections. The governing equations are solved for these nodal points. Several mobile-bed models are formulated using the finite element method. Generally, the application of this method is more efficient in two-dimensional modeling. A lot of sources and references describe the basic principles of the method for fluid mechanics and hydraulics, such as Conner and Brebbia (1976) and Katopodes (1984). Several mobile-bed and turbidity currents simulation models were proposed utilizing the finite element technique such as Choi and Garcia (1995), Choi (1998) and RMA2 WES of US Army (2001)

2-5-4-3 FINITE DIFFERENCE METHOD

Most of existing practical applications of mobile-bed modeling are based on the finite difference approach, which leads to the replacement of the governing equations, the differential form, by a system of algebraic equations. This may be done in two different ways: either by solving all equations simultaneously, coupled solution, or by separating the solution of the system of equations related to the liquid flow phase from the equation representing the sediment flow phase, uncoupled solution, as mentioned before.

2-5-5 FINITE DIFFERENCE SCHEMES

The finite difference schemes used in unsteady flow modeling may be grouped into several distinct classes according to their main features. In general, finite difference schemes are grouped into explicit and implicit schemes. The following subsections highlights on the various types of these schemes regarding one-dimensional models

2-5-5-1 EXPLICIT SCHEMESES

Explicit finite difference schemes are those in which the flow variables at any point j at the time level n+1 may be computed based entirely on known data at a few adjacent points at time level n. These schemes do not lead to a system of algebraic equations, since each point can be computed separately. Stoker, in 1957, was the first to use and introduce the explicit scheme for real life flood propagation. Some of such schemes are described hereafter;

2-5-5-1-1 LAX SCHEME

Considering the following differential system;

$$\frac{\partial f}{\partial t}+\frac{\partial B}{\partial x}=0 \quad \text{……………………………………………(2.5.5.1)}$$

The Lax scheme is applied to this general vector form of the homogeneous system of equation is based upon the following approximation of derivatives:

$$\frac{\partial f}{\partial t}\approx\frac{f_j^{n+1}-\left[\alpha f_j^n+(1-\alpha)\frac{f_{j+1}^n+f_{j-1}^n}{2}\right]}{\Delta t} \quad \text{…………………….(2.5.5.2)}$$

$$\frac{\partial B}{\partial x}\approx\frac{B_{j+1}^n-B_{J-1}^n}{2\Delta x} \quad \text{………………………………………(2.5.5.3)}$$

Where $0\le\alpha\le1$.

This scheme yields the exact solution of a fully linearized system of equations for a particular choice of the coefficient α and for $\Delta x/\Delta t$.

2-5-5-1-2 THE LEEP-FROG SCHEME

This scheme is the commonly used for numerical solution of the one-dimensional wave equations. In this scheme the derivatives are approximated as;

$$\frac{\partial f}{\partial t}\approx\frac{f_j^{n+1}-f_j^{n-1}}{2\Delta t} \quad \text{…………..…………………………(2.5.5.4)}$$

$$\frac{\partial B}{\partial x} \approx \frac{B_{j+1}^{n} - B_{j-1}^{n}}{2\Delta x} \qquad \text{..(2.5.5.5)}$$

A problem of computation at boundary points arises, as it does for all explicit schemes.

2-5-5-1-3 DELFT HYDRAULICS SCHEME

This scheme is based on the concept of nodes, or computational cells at the centre of which the dependent variables are computed. The system of equations, governing sediment transport phenomenon, is solved using the hypothesis that the two dependent variables h(x, t) and z(x, t) are computed in two separate steps. In the first step z is considered to constant during the time interval Δt and h(x, $t_n + \Delta t$) is computed from equation of motion using the following scheme

$$\frac{df}{dx} \approx \vartheta \frac{f_{j+1}^{n+1} - f_{j-1}^{n+1}}{2\Delta x} + (1-\vartheta)\frac{f_{j+1}^{n} - f_{j-1}^{n}}{2\Delta x} \qquad \text{..........................(2.5.5.6)}$$

Where, ϑ is a weighting coefficient ranging between 0.5 – 1. In the second step, the equation of sediment continuity is solved for z(x, t_{n+1}) using the known values of h(x, t_n) and the following explicit finite difference approximation:

$$\frac{\partial z}{\partial t} \approx \frac{1}{\Delta t}\left\{ z_j^{n+1} - \left[(1-\alpha) z_j^{n} + \alpha \frac{z_{j+1}^{n} + z_{j-1}^{n}}{2} \right] \right\} \qquad \text{.........................(2.5.5.7)}$$

Sediment rate derivative can be discretized as follows,

$$\frac{\partial G}{\partial x} \approx \frac{G_{j+1}^{n} - G_{j-1}^{n}}{2\Delta x} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(2.5.5.8)$$

Thus, the new values of z(x, t_{n+1}) are computed from the following equation:

$$z_{j}^{n+1} = (1-\alpha) z_{j}^{n} + \alpha \frac{z_{j+1}^{n} + z_{j-1}^{n}}{2} - \frac{\Delta t}{2\Delta x(1-p)} \left(G_{j+1}^{n} - G_{j-1}^{n} \right) \quad \ldots\ldots\ldots(2.5.5.9)$$

The above-mentioned method presents an important advantage that; if the equation representing the backwater curve is properly solved, it enables channel reaches with critical sections to be easily simulated.

2-5-5-2 IMPLICIT SCHEMES

The implicit scheme was developed as result of the time step restriction imposed in order to satisfy the Courant condition. Implicit finite difference schemes can be constructed in many different ways. Some of these schemes are described below

2-5-5-2-1 ABBOTT-INOESCU SCHEME

in this scheme the two dependent variables f(x, t) and u(t) are computed at different grid points. That is f may be computed at all even points and G are computed at all odd points. If the time and space derivative are approximated by this scheme as follows:

$$\frac{\partial u}{\partial t} \approx \frac{u_j^{n+1} - u_j^n}{\Delta t} \qquad \text{……………………………………..(2.5.5.10)}$$

$$\frac{\partial f}{\partial t} \approx \frac{1}{2}\left(\frac{f_{j+1}^{n+1} - f_{j+1}^{n}}{\Delta t} + \frac{f_{j-1}^{n+1} - f_{j-1}^{n}}{\Delta t} \right) \qquad \text{………………………...(2.5.5.11)}$$

$$\frac{\partial f}{\partial x} \approx \frac{1}{2}\left(\frac{f_{j+1}^{n+1} - f_{j-1}^{n+1}}{2\Delta x} + \frac{f_{j+1}^{n} - f_{j-1}^{n}}{2\Delta x} \right)$$

…………………………(2.5.5.12)

2-5-5-2-2 VASILLIEV SCHEME

This scheme is a fully implicit one in which both dependent variables are computed at all grid points. It uses the following approximation of time and space derivatives:

$$\frac{\partial f}{\partial t} \approx \frac{f_j^{n+1} - f_j^n}{\Delta t} \qquad \text{……………………………………..….(2.5.5.13)}$$

$$\frac{\partial B}{\partial x} \approx \frac{B_{j+1}^{n} - B_{j-1}^{n}}{2\Delta x} \qquad \text{…………………………………………(2.5.5.14)}$$

2-5-5-2-3 GUNARATNAM-PERKINS SCHEME

This scheme is a finite difference approximation in a linearized homogeneous characteristic form. The derivatives are replaced by

$$\frac{\partial f}{\partial t} \approx \frac{1}{6}\frac{f_{j-1}^{n+1} - f_{j-1}^{n}}{\Delta t} + \frac{2}{3}\frac{f_{j}^{n+1} - f_{j}^{n}}{\Delta t} + \frac{1}{6}\frac{f_{j+1}^{n+1} - f_{j+1}^{n}}{\Delta t} \qquad \text{...........(2.5.5.15)}$$

$$\frac{\partial B}{\partial x} \approx \frac{B_{j+1}^{n+1} - B_{j-1}^{n+1}}{2\Delta x} \qquad \text{.............................(2.5.5.16)}$$

2-5-5-2-4 PREISSMANN SCHEME

This a four point implicit scheme in which the dependent variables and its derivatives are discretized as follows:

$$f(x,t) \approx \frac{\theta}{2}\left(f_{j+1}^{n+1} + f_{j}^{n+1}\right) + \frac{1-\theta}{2}\left(f_{j+1}^{n} + f_{j}^{n}\right) \qquad \text{.................(2.5.5.17)}$$

$$\frac{\partial f}{\partial t} \approx \frac{1}{2}\frac{\left(f_{j+1}^{n+1} + f_{j}^{n+1} - f_{j+1}^{n} + f_{j}^{n}\right)}{\Delta t} \qquad \text{..............................(2.5.5.18)}$$

$$\frac{\partial B}{\partial x} \approx \frac{\phi}{\Delta x}\left(B_{j+1}^{n+1} - B_{j}^{n+1}\right) + \frac{(1-\phi)}{\Delta x}\left(B_{j+1}^{n} - B_{j}^{n}\right) \qquad \text{......................(2.5.5.19)}$$

Where θ, ϕ are weighting coefficients ranging between 0 – 1.

2-5-6 ALGORITHMS OF MODELING SYSTEMS

A convergent discretized form of the governing equations, together with appropriate boundary conditions, furnishes a system of algebraic equations in terms of the unknown variables at the time level t_{n+1}. The equations resulting from explicit schemes are decoupled, so the solution algorithm is simple since the unknown values at that time level may be computed separately for every computational grid point.

Implicit schemes lead to large systems of non-linear algebraic equations which only be solved by any of known successive iteration methods. The advantages of using implicit schemes are lost unless efficient methods for the solution of such systems are used. In order to solve a non-linear system, they should be linearized first and then be solved. Most of existing channel modeling systems are based on one of two solution techniques: iterative matrix method Gerald and Wheatly (1989) and double sweep method. The later method is much used in several modeling systems. In one-dimensional river model, a computational point of a model is not linked directly to all other points, but only to adjacent ones. In quasi-two-dimensional plain models, a cell may be linked to several neighboring cells, but still the total number is small. Thus, the matrix of the linear system of equations is spare.

A simplified system of Sogreah for the equations governing of water and sediment discharges in alluvial channels, equations (2.5.2.1) to (2.5.2.3), were given in Cunge et al (1980), as follows;

$$\frac{\partial}{\partial x}\left(\frac{Q^2}{2A^2}+gy\right)+gS_f=0 \qquad \text{.....................................(2.5.6.1)}$$

$$\frac{\partial z}{\partial t}+\frac{1}{(1-p)b}\frac{\partial G_s}{\partial x}=0 \qquad \text{.......................................(2.5.6.2)}$$

Application of Preissmann's four-point scheme and linearization of the resulting expressions leads to the following system of equations in $\Delta z_j, \Delta y_j,\ j=1,2,.......,m$:

$$\lfloor A_j \rfloor\{\Delta w_j\}+\lfloor B_j \rfloor\{\Delta w_{j+1}\}+\{C_j\}=0 \qquad \text{..................................(2.5.6.3)}$$

Where the unknown vector

$$\{\Delta w_j\} = \begin{Bmatrix} \Delta y_j \\ \Delta z_j \end{Bmatrix} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.5.6.4)$$

In which, $\lfloor A_j \rfloor$ and $\lfloor B_j \rfloor$ are 2×2 matrices and $\{C_j\}$ is a two-component vector. The system need two boundary conditions to be complete, then the double sweep method is used as a solution tool. A similar form of the above-simplified system of Sogreah is considered later for modeling an alluvial channel system.

2-6 MODELS LIMITATIONS AND IMPROVEMENT AREAS

Numerical models of mobile-bed evolution were first developed in the 1960s. Several models tried to simulate bed changes, especially sediment transport models. A lot of one-dimensional mobile-bed models have been developed. Recently, Singh et al (2004) developed a model simulating hydraulic and bed transients in alluvial rivers. In addition, Singh et al (2004) enumerated most of such models, CHAR1 (Perdreau and Cunge, 1971), CHAR2 (Cunge and Perdreau, 1973), HEC-6 (Thomas and Prasuhn, 1977), IALLUVIAL (Karim and Kennedy, 1982), FLUVIAL (Chang, 1982), GSTARS-2.1 (Yang, 1987), CHARIMA (Holly et al, 1989), CARICHAR (Rahuel et al, 1989), and SEDICOUP (Holly and Rahuel, 1990), FCM (Correia et al, 1992). Additional effort carried by researchers to develop and formulate mathematical models in order to simulate the morphological processes in alluvial channels is very wide that can not easily collected. Some of such effort is present as by Haag and Bedford (1971), Prandle and Crookshamk (1974), Chang and Hill (1976), Combs et al (1977), Kouwen et al (1977) Mc Anally (1984), Krishnappan (1985), Holly and Karim (1986), Lyn (1987), Zhang and Kahawita (1987), Zhang and Kahawita (1990), Ballamudi and Chaudary (1991), Hsu and Holly (1992), MIDAS (Niekerk et al, 1992) Ahmed (1994), Awulachew (1994), Belleudy (2000), Duan et al (2001), Belleudy and Sogreah (2001), Ahmed (2002), CCHE1D (Vieira and Wu, 2002), Papanicolaou (2004) etc. Table (2.6.1) represents a comparison of some one-dimensional mobile- bed models.

Table (2.6.1) Some Mobile-Bed models

Model	Bed Load Predictor	Roughness predictor	Model Type
IALLUVIAL (1982)	Empirical formula	Empirical formula	Uncoupled
CARICHAR (1989)	Meyer-Peter& Muller	Manning-Strickler formula	Coupled
CHARIMA (1989)	Ackers & White	Manning formula	Coupled
MIDAS	Bridge &	Manning formula	Uncoupled

(1992)	Dominic		
Ahmed (1994)	Empirical formula	Empirical formula	Uncoupled
Awulachew (1994)	Several formulae	Manning formula	Coupled
HEC-RAS (1998)	Several formulae	Several formulae	Coupled
CCHE1D (2002)	Several formulae	Manning formula	Uncoupled
Singh et al (2004)	Several formulae	Several formulae	Coupled

It is clearly presented from the literature surveyed on numerical modeling of mobile-bed channel and the physics of the related morphological processes that, there some points need to be more explored and some research areas should be investigated in more depth. In addition, the main assumptions and considerations upon which the alluvial models are based need to be revised from the basic principles.

Some areas need to be improved that concerned with the energy slope as a driving parameter in mobile-bed models. A lot of formulae were suggested in the literature to compute the energy slope, however still most of the models computations are inconsistent due that parameter. Some great previous efforts on this part were elaborated, such as that presented by Chow (1959) on the internal energy losses and losses due to external forces, but Chow's work need to be continued to investigate the conceptual ideas that can be extended to movable boundary channels in order to derive an alluvial flow equation. Another point is that related to the energy consumed in transporting the sediment particles on the bed as assumed by some researches. Up to now there is specific formula to compute that part of energy.

Indeed, mobile-bed models are confronted by three principal difficulties:

1- Sediment transport predictors and the roughness relations give unreliable prediction.

2- Sediment transport mechanisms are often quite simplified in modeling system.

3- Numerical solution algorithms are often quite crude, introducing errors and possibly leading to marginally stable results.

Even today such mobile-bed models have not attained the degree of reliability and efficiency of fixed-bed models. Thus, to propose new theoretical relations, or to derive other formulae, related to flow in alluvial channels, on conceptual basis or even the development of more numerical models becomes a necessary statement. Thus, the following part of this study elaborates the approach and methodology followed to investigate the mentioned research areas and to derive new concepts, starting from the basic principles to be used in the development of a mobile-bed model.

CHAPTER FOUR

RESULTS AND ANALYSIS

4-1 THEORETICALLY PROPOSED ALLUVIAL FLOW EQUATION

In order to specify the values of the modification factors of the theoretically proposed alluvial flow equation, represented by equations (3.2.2.1), (3.2.2.2) and (3.2.2.3), experimental flow data or field data is required. The validity of the proposed equation has been investigated using SAFL experimental data, which was collected by a multi-university research-team at St. Anthony Falls Laboratory (SAFL). Cui et al (1996) described the experimental work and presented the verification of their numerical model carried on aggradations and downstream fining through these experiments. In addition, Belleudy and Sogreah (2001) used SAFL experiments in their numerical simulation of sediment mixture deposition.

4-1-1 SAFL EXPERIMENTS

A multi-university research team designed and performed a series of large-scale experiments at St. Anthony Falls Laboratory on downstream fining. Six experiments were performed in a channel at SAFL with a depth of 1.83 m, a width of 2.74m and a length of 60 m. Runs 1,2 and 3 were conducted using a narrower width of 0.305 m. Water discharge, sediment feed rate and the tailgate elevation were all kept constant during the run. The feed material was poorly

sorted, with geometric mean size 4.63 mm and specific gravity 2.65. Upon commencement of the sediment feed, a mildly concave aggradational wedge ending in a front was prograded downstream as shown below in figure (4.1.1). The runs were not continued when the front of sediment arrived a point between 35 and 40 m downstream of the feed point.

SAFL experimental data presented in a series of runs, for each run the discharge, Q, the sediment feed rate, Q_s, the migration length, L_f, the duration of the experiment, T, and the tailgate water surface elevation, z_t, are given in table (4.1.1). As stated earlier, in alluvial boundary channels, the resistance due to the form drag is usually added to the resistance due to the skin friction in order to evaluate the roughness coefficient. Herein, Strickler roughness coefficient was taken as given by Belleudy and Sogreah (2001). The bed slope was considered as the slope of the deposited sediment at the end of the simulation time.

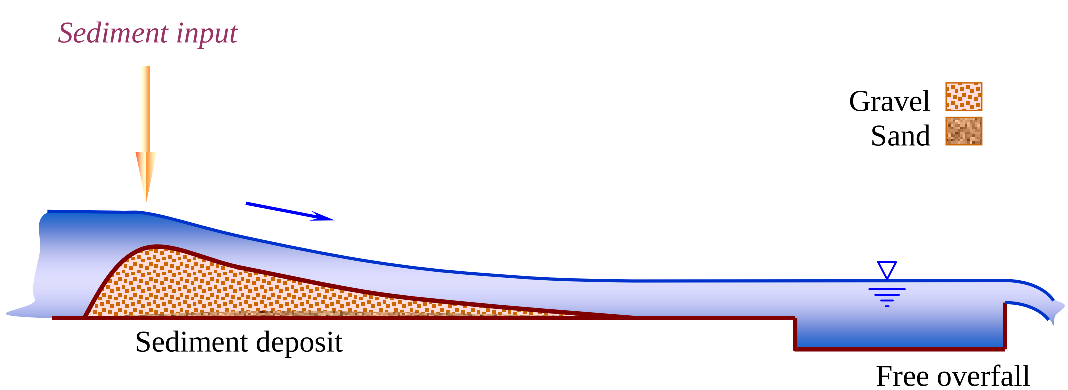

Fig. (4.1.1) Schematic Diagram of the configuration of SAFL Runs

Table (4.1.1) SAFL Experiments Data

Run	1	2	3
Q (lit. /sec)	49	49	49
Q_s (kg/min)	11.30	5.65	2.83
L_f (m)	38	38	35
T (hr)	16.83	32.4	64
Z_t (m)	0.40	0.45	0.50

{Source: Cui et al (1996)}

4-1-2 NUMERICAL TESTS

Theoretically proposed alluvial flow equation was tested in a manner that, the proposed modification factors incorporated in the equation to compute the flow surface profile in an alluvial channel. A series of numerical tests utilizing the described SAFL experiments were carried on in order to examine the effect of each modification factor on the proposed equation. The selection of the values of the modification factors was considered to be less than unity since the normal flow depth, the critical flow depth and the water slope in an alluvial channel are assumed to be reduced. This

implies that an optimum set of values for the modification factors has to be searched for in order to satisfy the cases of SAFL experiments. Values ranging between 0.9 and 0.1 were tested for the proposed modification factors for Run 1, as a calibration run. Each set of values of the modification factors was applied into the modified gradually varied flow equation in order to reach to the optimum set of values that satisfies the measured water surface profile in Run 1. A series of different water surface profiles were computed corresponding to each set of values of modification factors.

The effect of variation of the values of each modification factor was examined through the calibration run. The effect of varying the normal depth modification factor on the computation of the water surface profile was studied. Figure (4.1.2) and figure (4.1.3) show the effect of this variation on SAFL experiments. The series 1 and 2 show the measured water level and the measured bed level while series 3, 4 and 5 represent the computed water level using the modified gradually varied flow equation corresponding to the Normal Depth Modification Factor (N.D.M.F.) values 0.8, 0.7 and 0.6 respectively.

The variation in values of the critical depth modification factor was examined. This variation is shown in figure (4.1.4) and figure (4.1.5). The series 3, 4 and 5 represent the computed water levels corresponding to the Critical Depth Modification Factor (C.D.M.F.) values 0.7, 0.5 and 0.3 respectively. Finally, the effect of the variation of the water surface slope modification factor was investigated. Figure

(4.1.6) and figure (4.1.7) represent the effect that variation. The series 3, 4 and 5 represent the computed water levels corresponding to the Water Surface Modification Factor (W.S.M.F.) values 0.5, 0.3 and 0.1 respectively. It is clearly shown that the water slope modification factor is the more effective parameter.

Thoroughly investigated numerical tests and deep analysis of the effect of the variation of the values of the modification factors on the data of Run 1 showed the optimum set of values of the proposed modification factors that satisfies the case examined, so that the computed water level is correctly predicted, is not easy to be determined by trial and error but could be obtained following one of the optimization techniques. Thus, the conceptually derived alluvial flow equation is the easier to be verified.

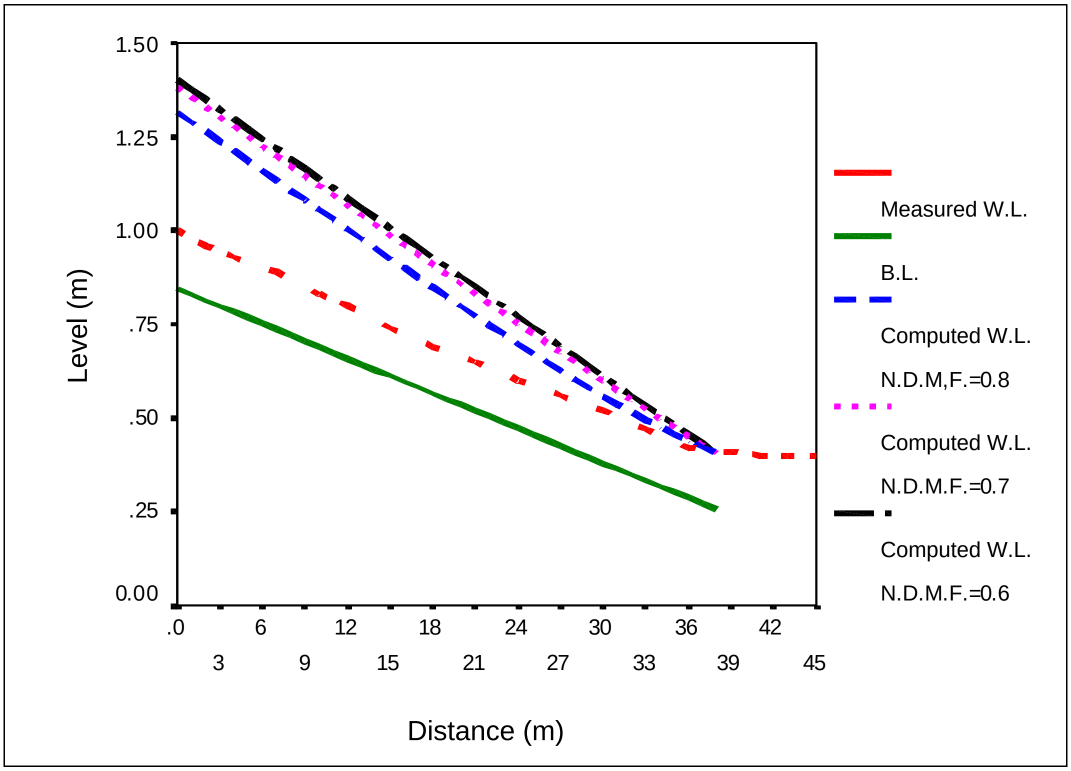

Fig. (4.1.2) Effect of Variation of The Normal Depth Modification Factor values with β =0.7 and λ =0.7

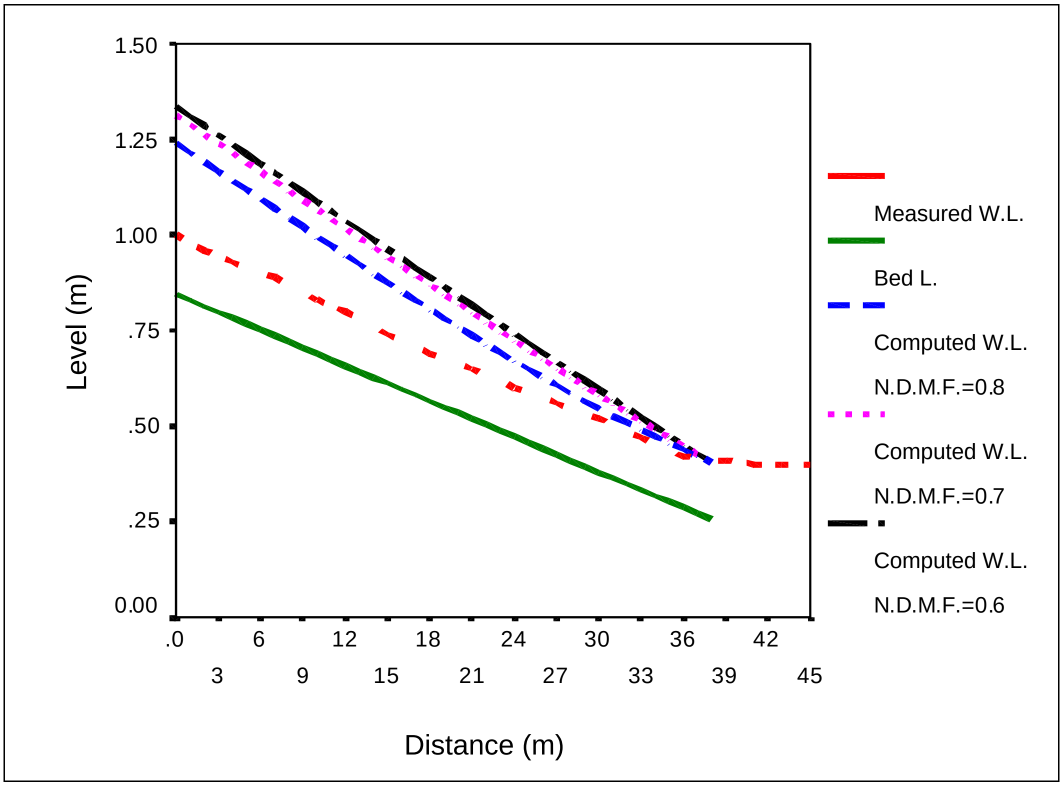

Fig. (4.1.3) Effect of Variation of The Normal Depth Modification Factor values with β=0.6 and λ=0.6

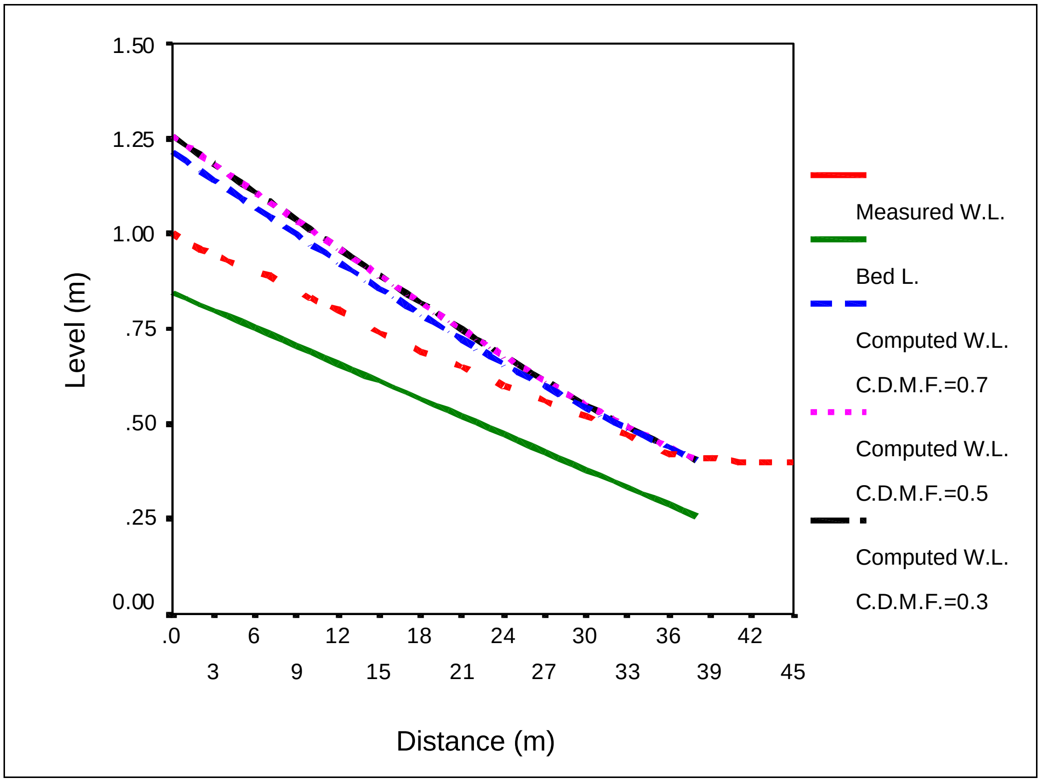

Fig. (4.1.4) Effect of Variation of The Critical Depth Modification Factor values with α =0.8 and λ =0.6

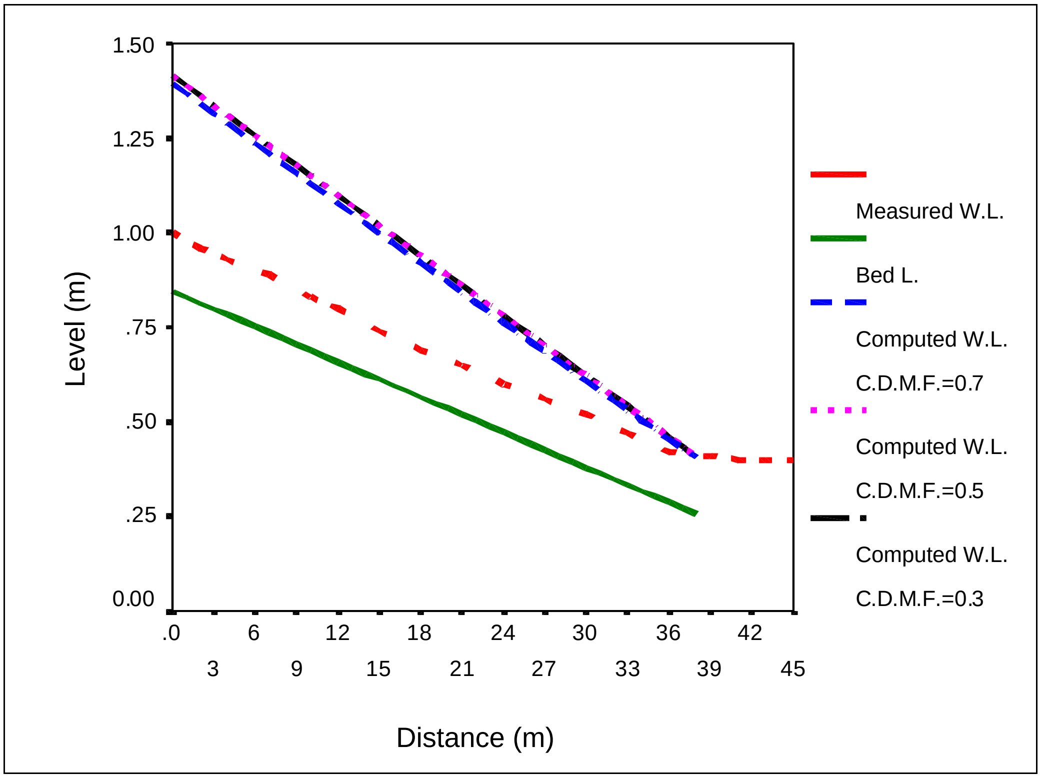

Fig. (4.1.5) Effect of Variation of The Critical Depth Modification Factor values with α =0.5 and λ =0.7

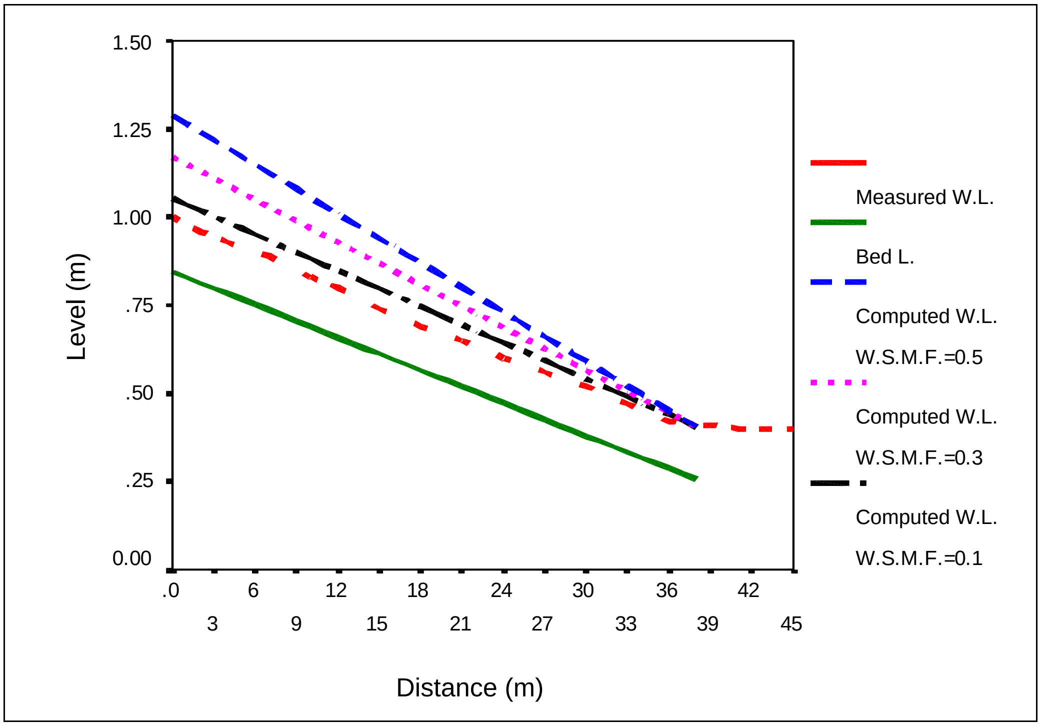

Fig. (4.1.6) Effect of Variation of The Water Slope Modification Factor values with α =0.5 and β =0.6

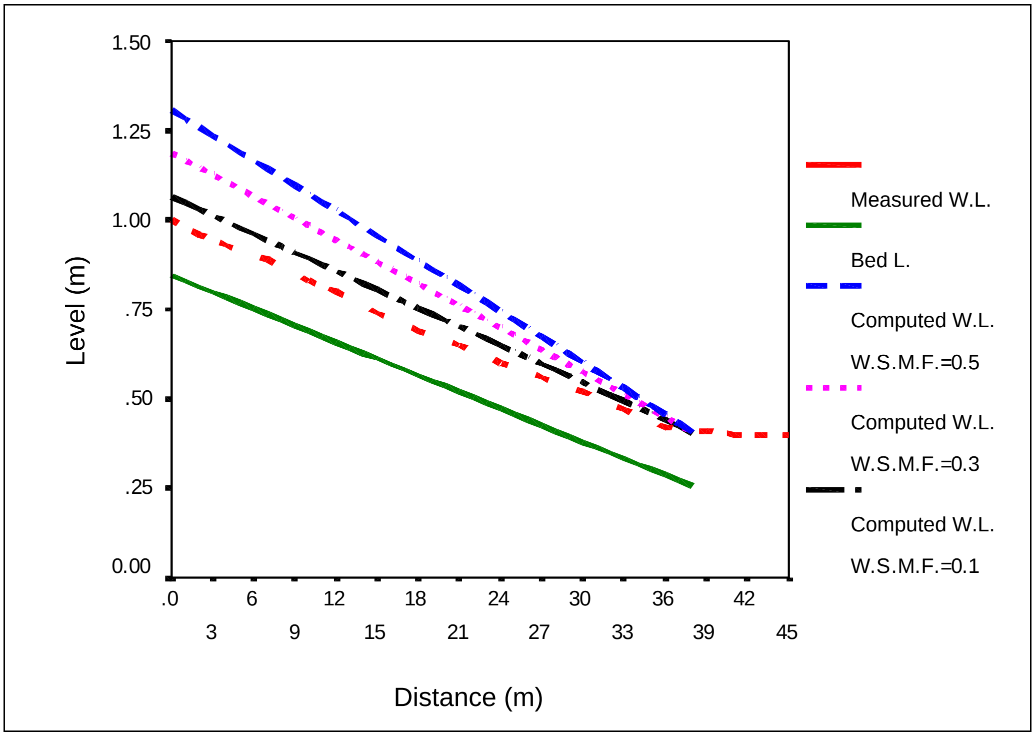

Fig. (4.1.7) Effect of Variation of The Water Slope Modification Factor values with α =0.2 and β =0.7

4-2 CONCEPTUALLY DERIVED ALLUVIAL FLOW EQUATION

The sediment characteristic parameter, φ, is a function of the sediment coefficient, ϕ, and the specific gravity, ss, as shown earlier in equation (3.3.3.14). The sediment coefficient considered to be related to the saturated angle of repose of the sediment particles and the size distribution. More investigations were carried using previously described SAFL experimental data to clearly specify that relation. The saturated angle of repose of the sediment material is considered to be θ. A study, in a sensitivity analysis form, was made assigning a range of values for the saturated angle of repose between 2 - 10 degrees, as shown in table (4.2.1). The sediment coefficient was assumed to be $\varphi = \tan\theta$. The corresponding values of the sediment characteristic parameter were calculated from equation (3.3.3.14) with the specific gravity value 2.65 for SAFL data. On the other hand, the corresponding values of the critical depth modification factor, β, and the water slope modification factor, λ, were computed from equation (3.3.3.19) and equation (3.3.3.20) respectively.

Table (4.2.1) Sensitivity analysis parameters

θ	ϕ	φ	β	λ
10.0	0.18	3.1	0.69	0.65
7.5	0.13	3.9	0.64	0.51
6.0	0.10	4.8	0.59	0.42
5.0	0.08	5.7	0.56	0.35
3.0	0.05	8.6	0.49	0.23
2.5	0.04	10.4	0.46	0.19
2.0	0.03	13.6	0.42	0.15

The effect of variation of the values of the of the sediment coefficient, ϕ, was examined through SAFL data. The computed values of the sediment characteristics parameter, φ, corresponding to each value of ϕ, were substituted to test the validity of the conceptually derived alluvial flow equation, equation (3.3.3.17), using the calibration run. Figures (4.2.1) through (4.2.6) represents the variation in the computed water level according to change in the values of ϕ. It is clearly shown that, as θ, assumed saturated angle of repose, decreases the computed water level be closer to the measured water level. After certain value of

θ, no effective difference is noticed. A value of ϕ equal 13.6 was used to verify the derived alluvial flow equation for the other two runs of SAFL data. This value is considered to correspond to an optimum set of modification factors that satisfies the calibration run. The computed optimum set is as follow:

$$\alpha = 0.812$$
$$\beta = 0.42$$
$$\lambda = 0.15$$

Figure (4.2.7) and figure (4.2.8) represents the application of the conceptually derived alluvial flow equation to the other two verification runs, run 2 and run 3 respectively. Using these results, it can be stated that the values of the sediment coefficient, ϕ, could be taken more or less in the range of values given in table (4.2.1), according to properties of the sediment in the alluvial channel.

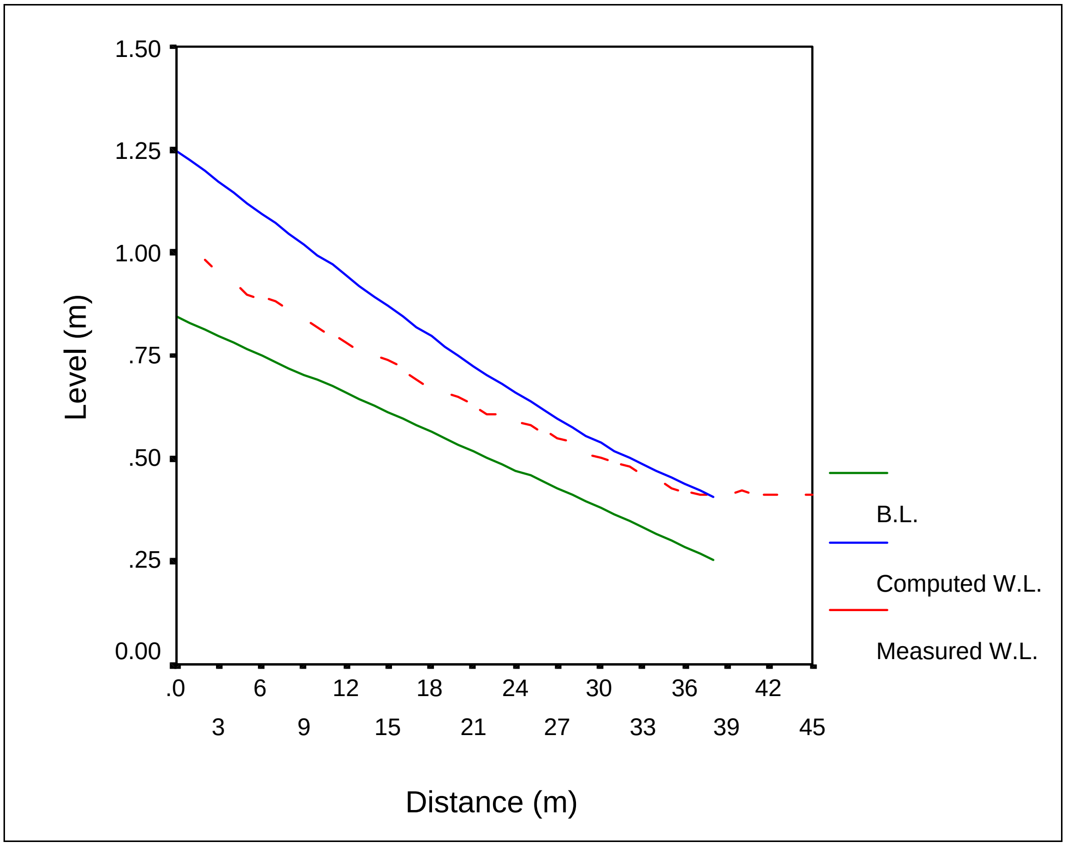

Fig. (4.2.1) Computed Water Level (ϕ=0.13)

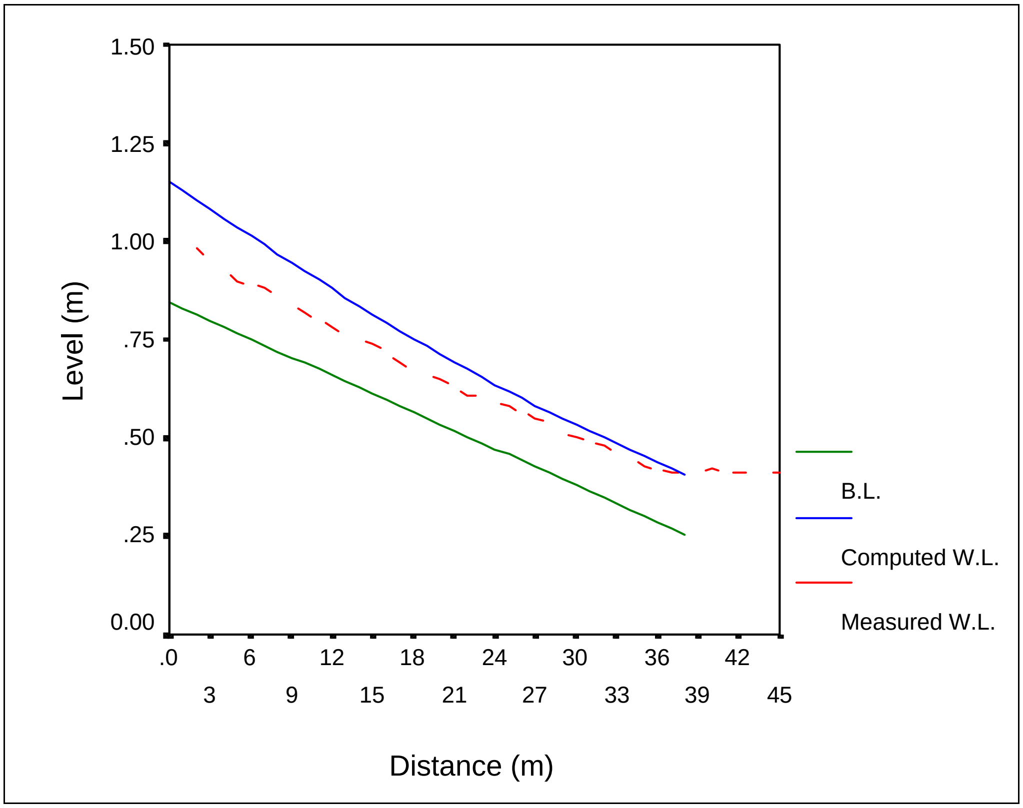

Fig. (4.2.2) Computed Water Level (ϕ=0.10)

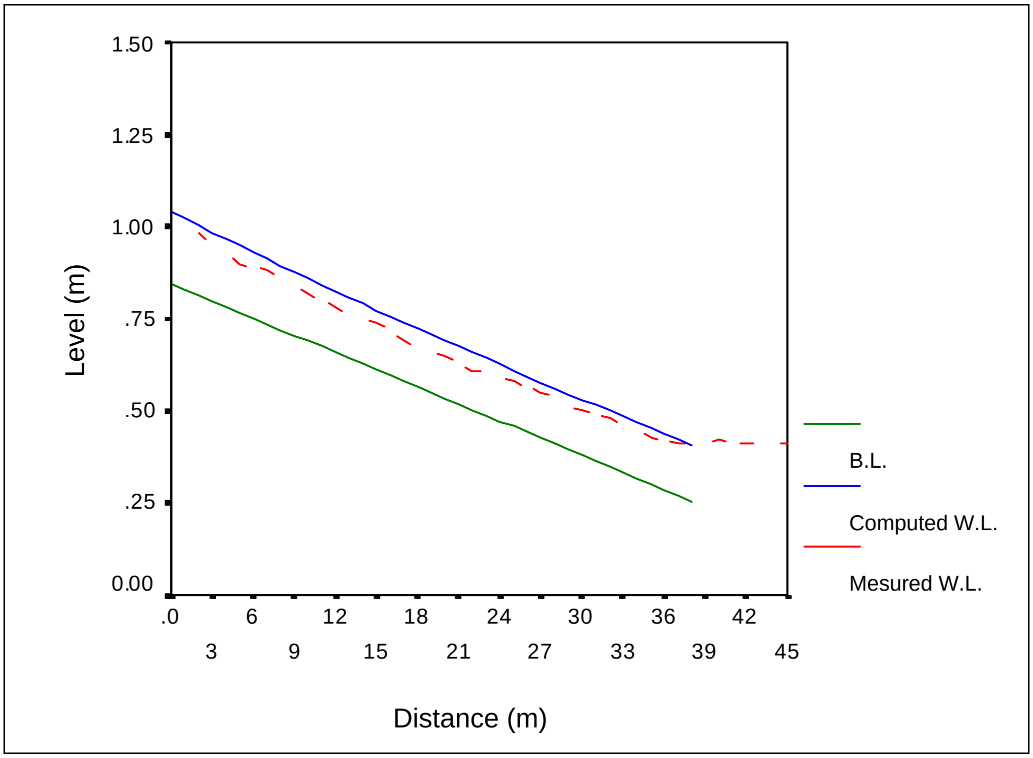

Fig. (4.2.3) Computed Water Level (ϕ=0.08)

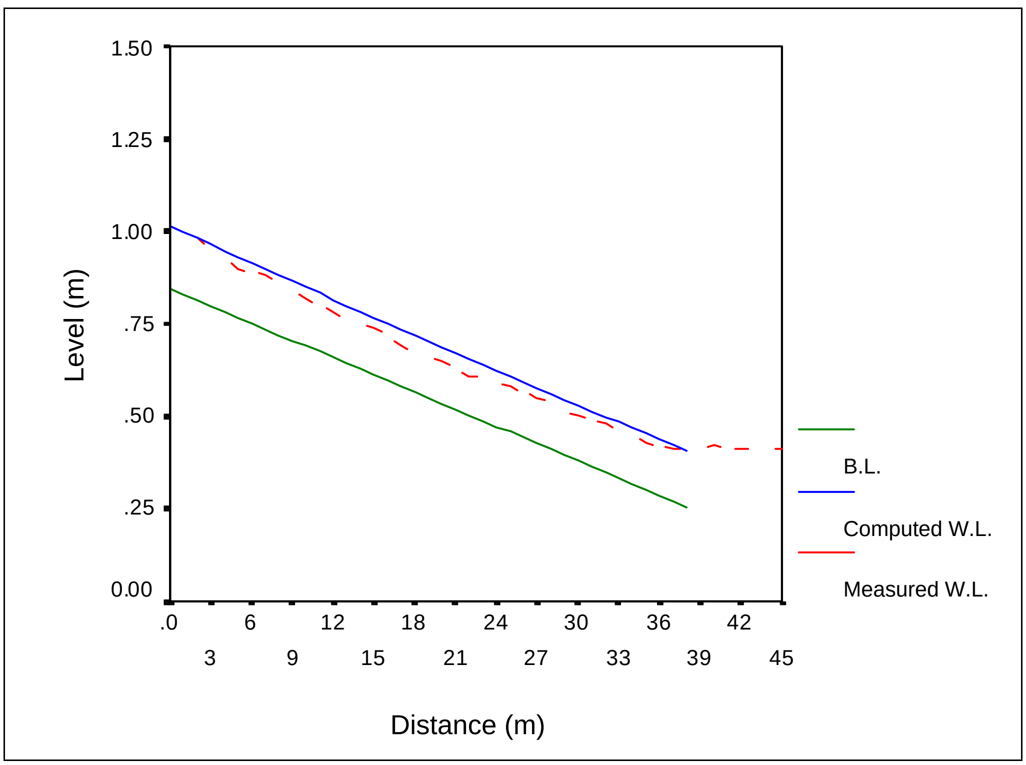

Fig. (4.2.4) Computed Water Level (ϕ=0.05)

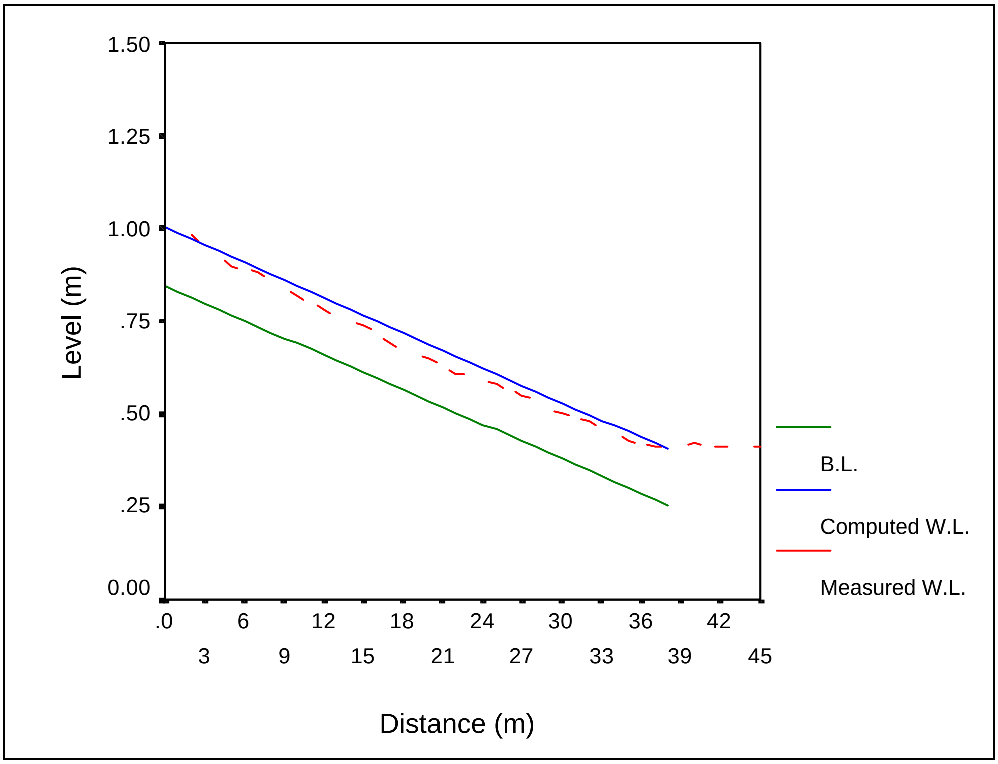

Fig. (4.2.5) Computed Water Level (ϕ=0.04)

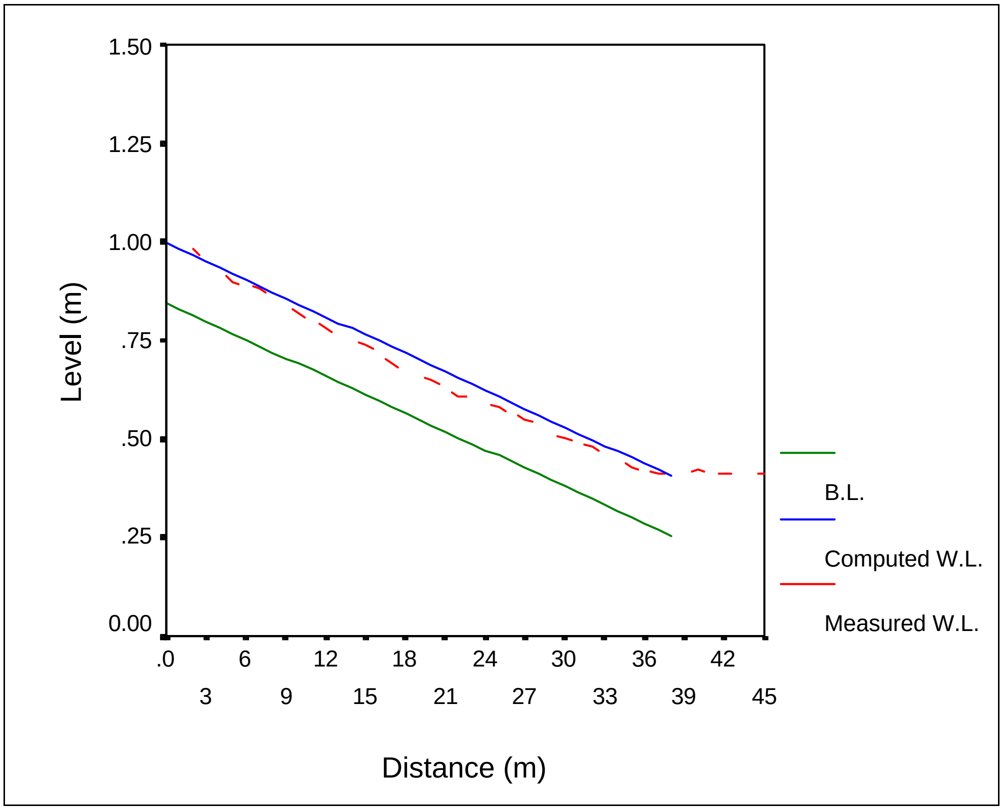

Fig. (4.2.6) Computed Water Level (ϕ=0.03)

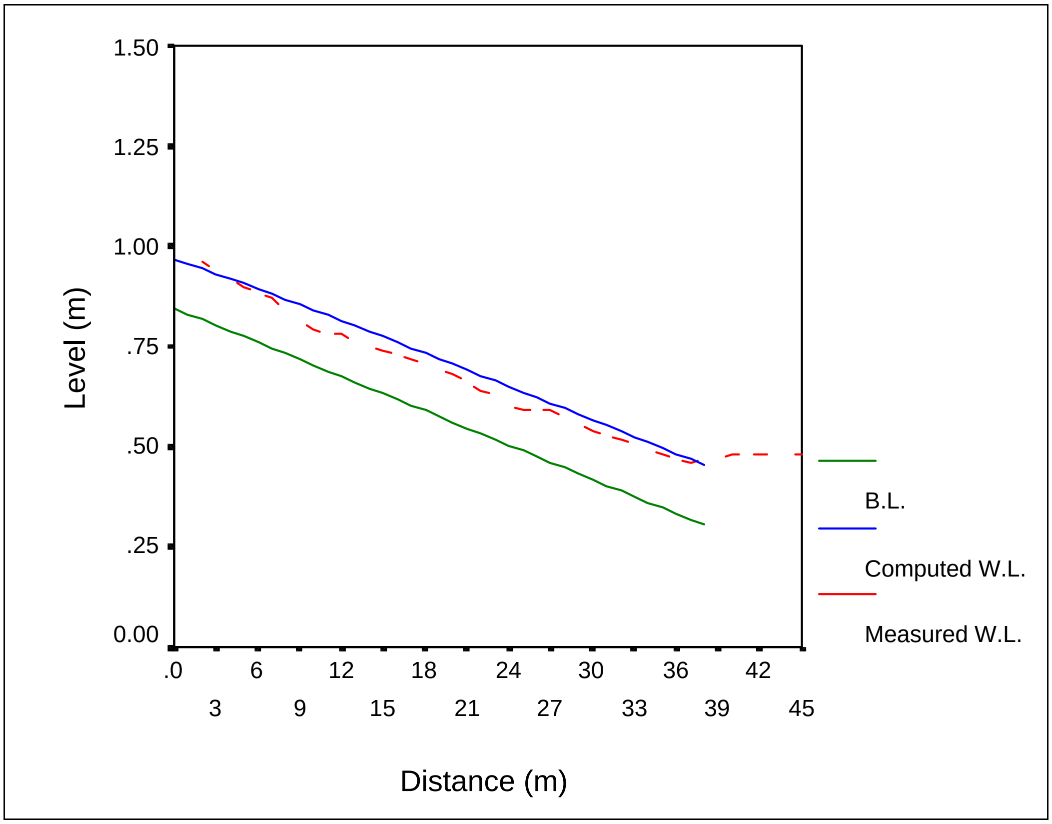

Fig. (4.2.7) Computed Water Level Verification Run 2

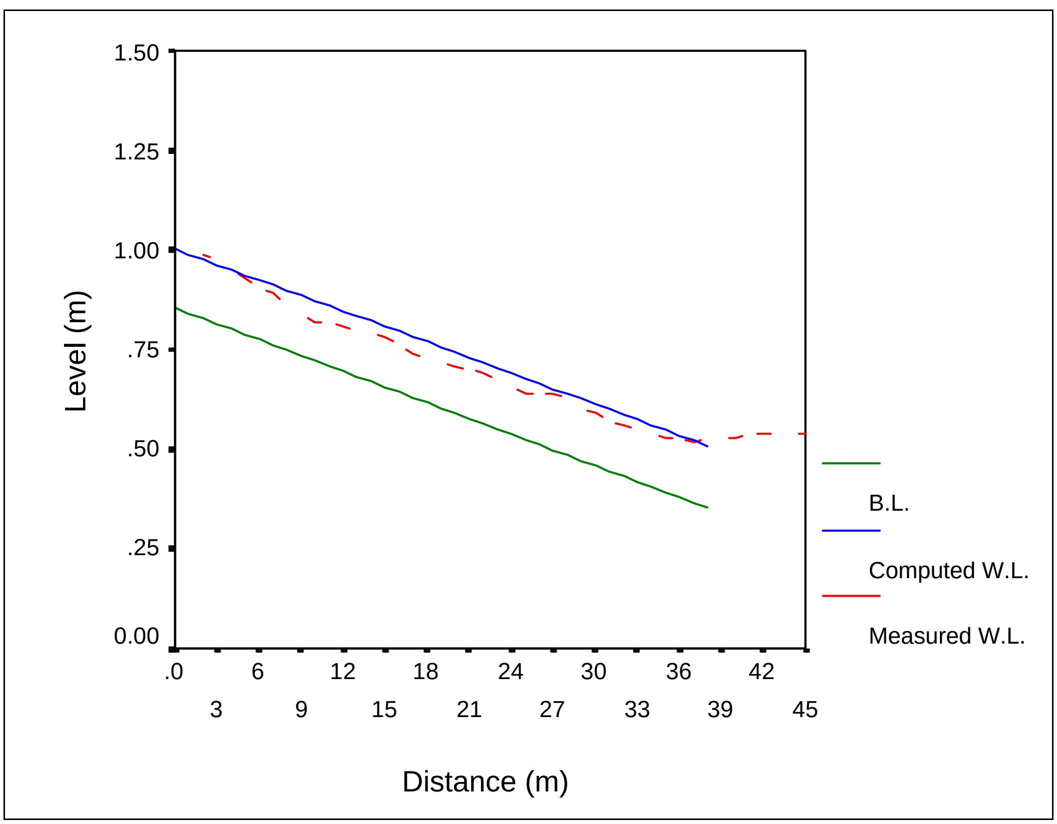
1.50
1.25
1.00
.75
.50
.25
0.00
Level (m)
.0
3
6
9
12
15
18
21
24
27
30
33
36
39
42
45
Distance (m)
B.L.
Computed W.L.
Measured W.L.

Fig. (4.2.8) Computed Water Level Verification Run 3

4-3 CALIB RATION OF SEDTREN MODEL

The available field data for an alluvial reach, mentioned by other investigators, was used in this research to calibrate the newly developed SEDTREN model. A schematic river reach having the overall hydraulic and sediment characteristics of the lower Rhone river, in France, as described by Gemaehling et al in 1957, used by Mohammed-Abdalla et al (1986) and Rahuel et al (1989) to develop their models concerning the simulation of non-uniform bed load transport in alluvial channels.

4-3-1 RAHUEL'S CARICHAR MODEL

Rahuel et al (1989) described CARICHAR model, a numerical model treats bed load transport of non-uniform sediment mixture solved with a coupled implicit manner using the Preissmann finite difference scheme. In the model two equations were adopted to determine the bed load transport, Mayer-Peter and Muller Relation and loading law. The spatial delay effects the non-equilibrium bed load transport are taken into account through use of a developed notation of a loading law. This loading law is considered to count for the difference between the field-scale conditions and the steady-flow experiments from which the bed load predictors are developed, thus to take into account the spatial bed load delay.

The details of the technique and the code of CARICHAR model are given in Rahuel's, Ph.D. thesis, (1988). Implementation of the model is demonstrated through application to a schematic reach of Rhone River in France. The total length of river modeled is 38 km, and the initial constant bed slope 0.0007. The initial bed-material distribution is shown in fig. (4.3.1).

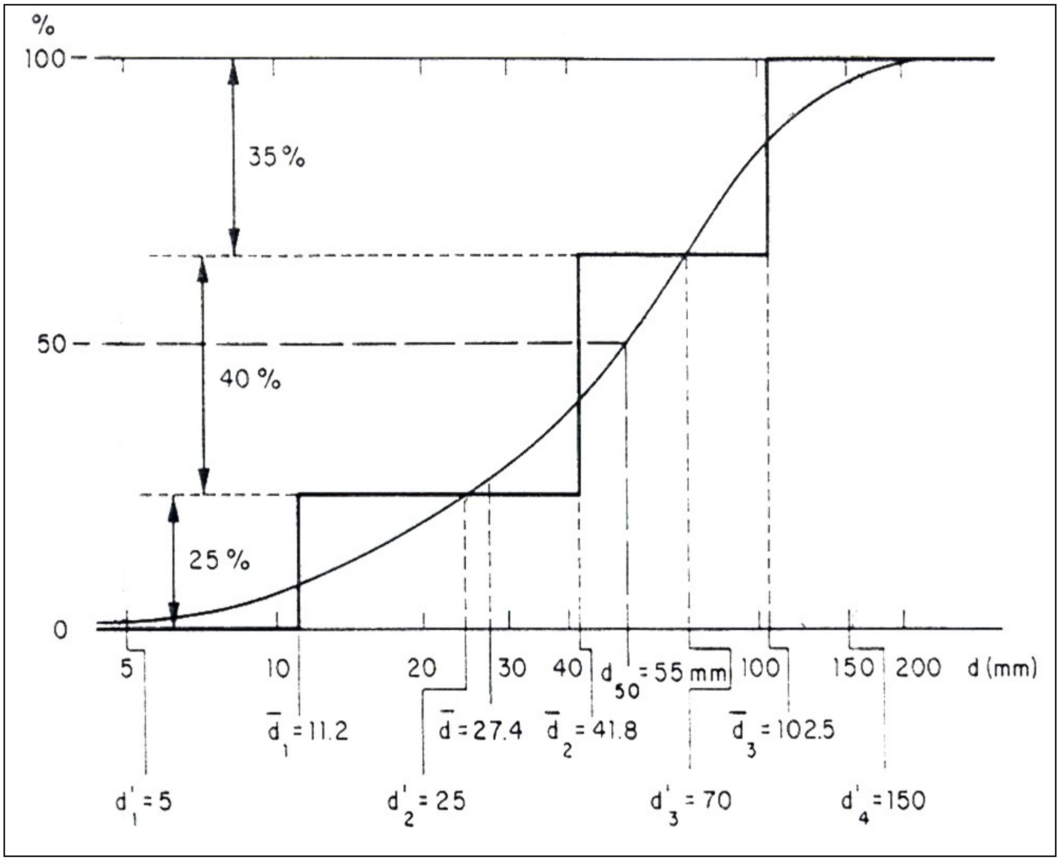

Fig. (4.3.1) Initial Bed-Material Distribution (Rhone River)
{Source: Rahuel et al (1989)}

The characteristic diameter of a class was taken as the geometric mean of the two diameters delimiting the class. The spatial discretization of the model tested is two kilometers distant between the computational points with a time step of 48 hours. The tests were based on a dam that raises the water level by 10 meters compared to its uniform-flow value, while a constant discharge of 4000 m^3/sec enters the river at its upstream limit.

Fig. (4.3.2) shows the results of a simulation of 720 days with a single sediment particle size class and using the loading law of Bell and Sutherland loading law, based on comprehensive analysis of laboratory tests. The law is given as follows:

$$\frac{\partial q_s}{\partial x} = K_l\left(q_s^* - q_s\right) + \frac{q_s}{q_s^*}\frac{\partial q_s^*}{\partial x} \qquad \text{.....................................(4.3.1.1)}$$

In which q_s is the sediment transport rate per unit width; q_s^* is the equilibrium value of q_s; and K_l is a loading-law coefficient. This law introduces a spatial delay only if the sediment transport is different from its equilibrium value. In this simulation the deposited delta has an abrupt leading edge. As mentioned by Rahuel et al (1989), this physically reasonable behavior is observed whatever the value of the loading law coefficient. Figure (4.3.3) shows the results for the same reach with other loading law, Daubert and Labreton, which introduces a systematic spatial delay between the sediment transport rate and its equilibrium value. This simulation showed that the deposited delta formed in the reservoir is quite thin and it spreads fairly rapidly downstream. The solid discharge longitudinal profiles for the two situations are shown in figure (4.3.4) and figure (4.3.5).

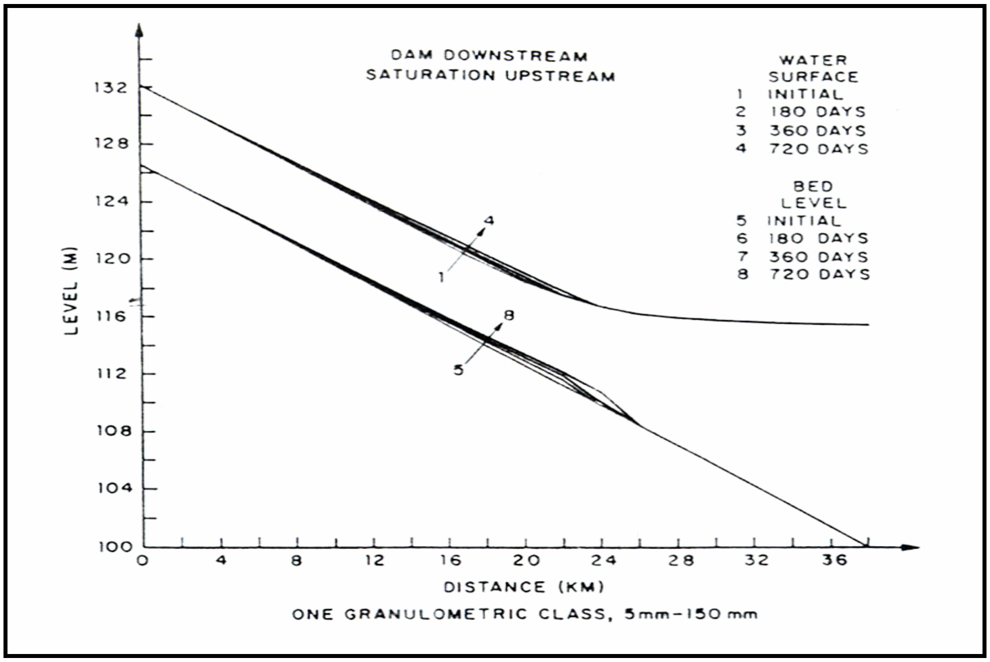

Fig. (4.3.2) CARICHAR Simulation Using Bell and Sutherland loading law.

{Source: Rahuel et al (1989)}

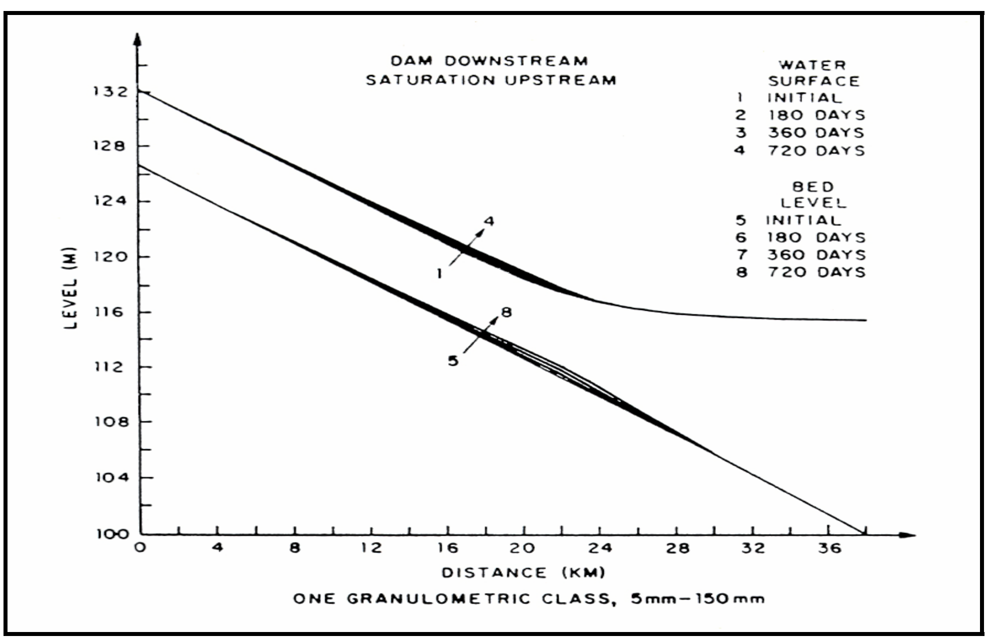

Fig. (4.3.3) CARICHAR Simulation Using Daubert and Labreton loading law.

{Source: Rahuel et al (1989)}

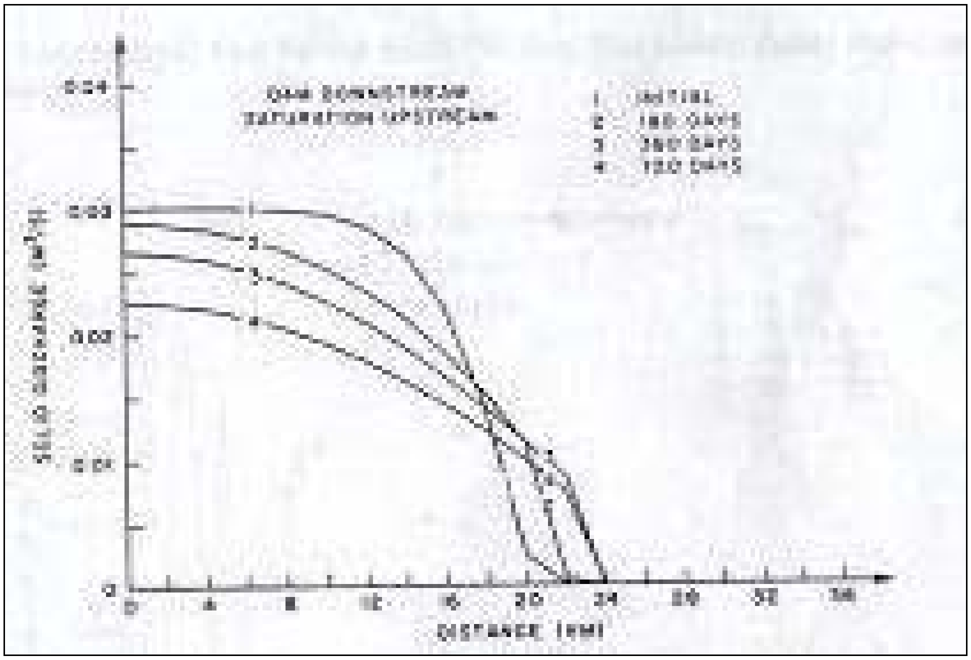

Fig. (4.3.4) Sediment Transport by Bell and Sutherland loading law

{Source: Rahuel et al (1989)}

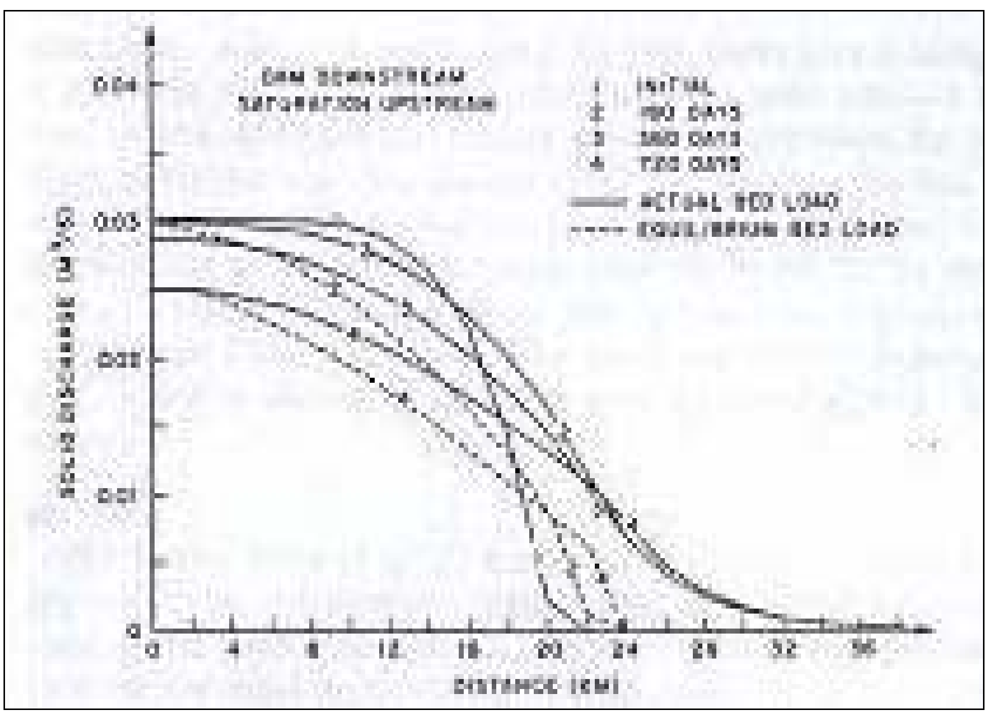

Fig.(4.3.5) Sediment Transport by Daubert and Labreton loading law
{Source: Rahuel et al (1989)}

4-3-2 PARAMETRIC EVALUATION OF THE MODEL

To calibrate the developed model, SEDTREN model was implemented to the Rhone River reach data utilizing the results presented by Rahuel et al (1989). The same reach simulated with various values of $\Delta t/\Delta x$. The spatial discretization of the tested channel reach one kilometer distant between the computational points with a time step of 48 hours kept as modeled by Rahuel were found to be a suitable $\Delta t/\Delta x$. The sediment properties incorporated in this formulation are same as those mentioned by Rahuel's formulation. Three classes for the sediment size of geometric means 11.2mm, 41.8mm and 102.5mm with percentage distributions 25%, 40% and 35% respectively have been used as the representative particles diameters of the non-uniform sediment mixture.

To study the effect of the dimensionless shear stress parameter, introduced in the proposed bed-load predictor equation (3.4.1.2), on the model, various values were assigned to simulate the spatial bed evolution of the reach. Longitudinal profiles of Rhone River reach were also simulated representing the rate of the transported bed load. In addition, the temporal evolution of the channel reach was carried out for different values of the dimensionless shear stress parameter. Several values for the dimensionless shear stress parameter ranging between 0.04 and 0.18 were used to simulate the case. The values of the shear stress parameter are chosen in that range, as it is the range used by most of similar sediment transport predictors. On the other hand, the values of the sediment coefficient ranged between 0.015 and 0.15, a value of 0.03 was used to simulate the bed evolution and the sediment transport rate. The developed transporting energy concept was introduced in a graphical representation using a specified value for the shear stress parameter. The applications of the mentioned model parameters are elaborated in the Next chapter.

CHAPTER FIVE

MODEL APPLICATIONS

5-1 EVALUATION OF THE MODEL PARAMETERS

To evaluate the model parameters, parametric analysis for SEDTREN model was carried, simulating the reach of the Rhone River modeled previously by CARICHAR model. The calibration of the formulated model includes studying the effect of the main parameters, the dimensionless shear stress parameter and the sediment coefficient. The diameter of the formed armor layer and the new sediment particle distribution of the active layer were also computed. The flow characteristics represented in the dimensionless numbers, Froude number and the Reynolds particles number were computed along the channel reach.

5-2 DIMENSIONLESS SHEAR STRESS PARAMETER

To study the effect of the dimensionless shear stress parameter on the model, various values were assigned to simulate the spatial bed evolution of the reach. Longitudinal profiles of Rhone River reach were also simulated representing the rate of the transported bed load. In addition, the temporal evolution of the channel reach was carried out for different values of the dimensionless shear stress parameter. The developed concept of transporting energy line was introduced in a graphical representation using a specified value for the dimensionless shear stress parameter.

5-2-1 SIMULATION OF SPATIAL BED EVOLUTION

The bed evolution along the simulated channel reach was modeled using various shear stress parameter values in SEDTREN mobile-bed models. Figure (5.2.1), figure (5.2.2), figure (5.2.3), figure (5.2.4) and figure (5.5.5) represent the effect of the dimensionless shear stress parameter with values equal to 0.04, 0.09, 0.15, 0.16 and 0.18 respectively. Using a value of 0.18 showed the same delta situation given by CARICHAR model.

It is shown that for the modeled reach, as the value of the shear stress parameter become smaller the delta shape become larger and takes irregular from as simulation time changes. Whereas the delta shapes seem to be more uniform and flatter with simulation time as the shear stress parameter become larger. Furthermore, it is noticed that, the deposited sediment configures the delta shape at positions varied with the value of the dimensionless shear stress parameter. The simulated delta formation is shown at a distance upstream as the chosen value of the shear stress becomes smaller, and the simulated delta formed downstream as the value becomes larger. In addition, it is clear that as the dimensionless shear stress parameter become smaller the deposited sediment volume is larger while the deposited volume become smaller when the parameter is larger.

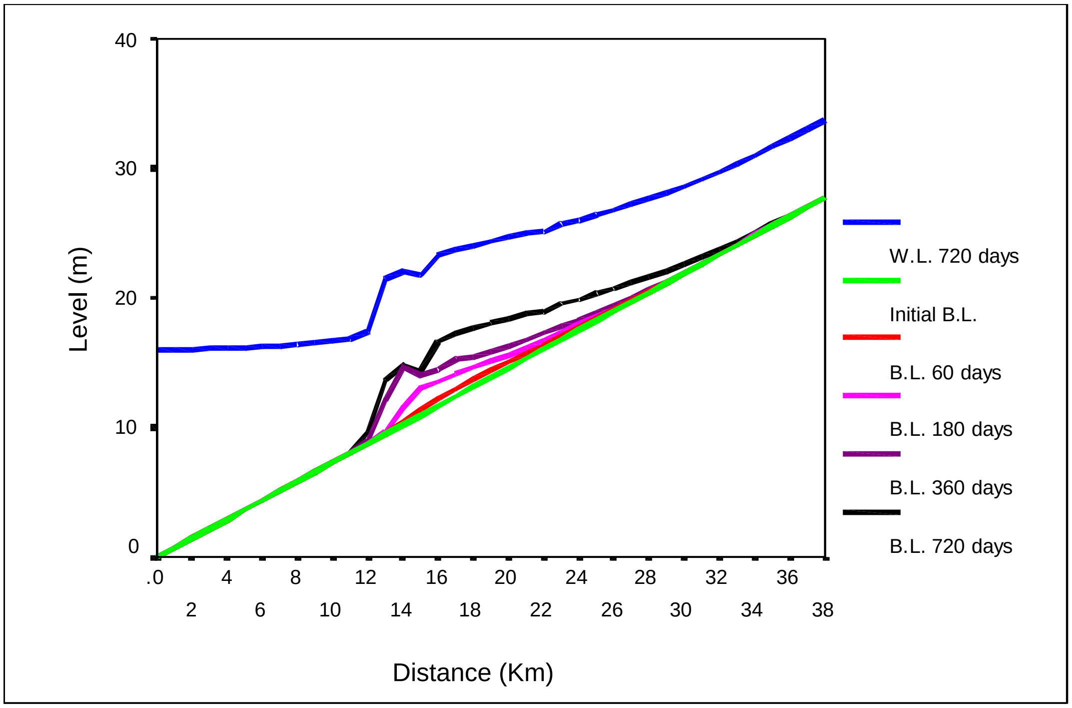

Fig. (5.2.1) Simulation of Bed Evolution (Shear Stress Parameter=0.04)

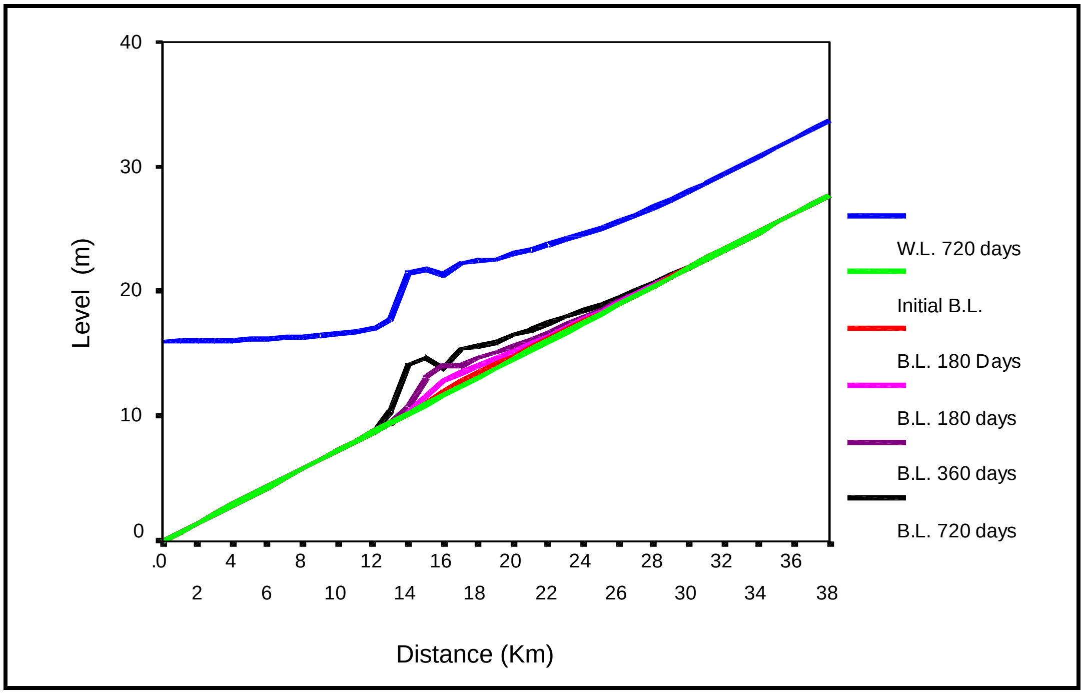

Fig. (5.2.2) Simulation of Bed Evolution (Shear Stress Parameter=0.09)

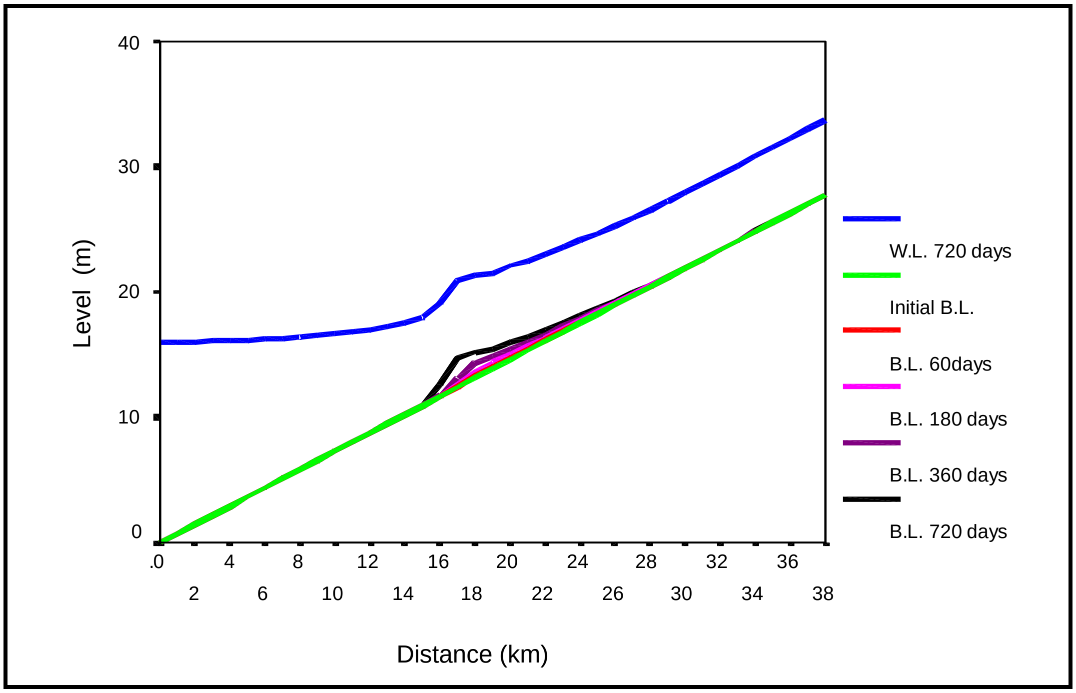

Fig. (5.2.3) Simulation of Bed Evolution (Shear Stress Parameter=0.15)

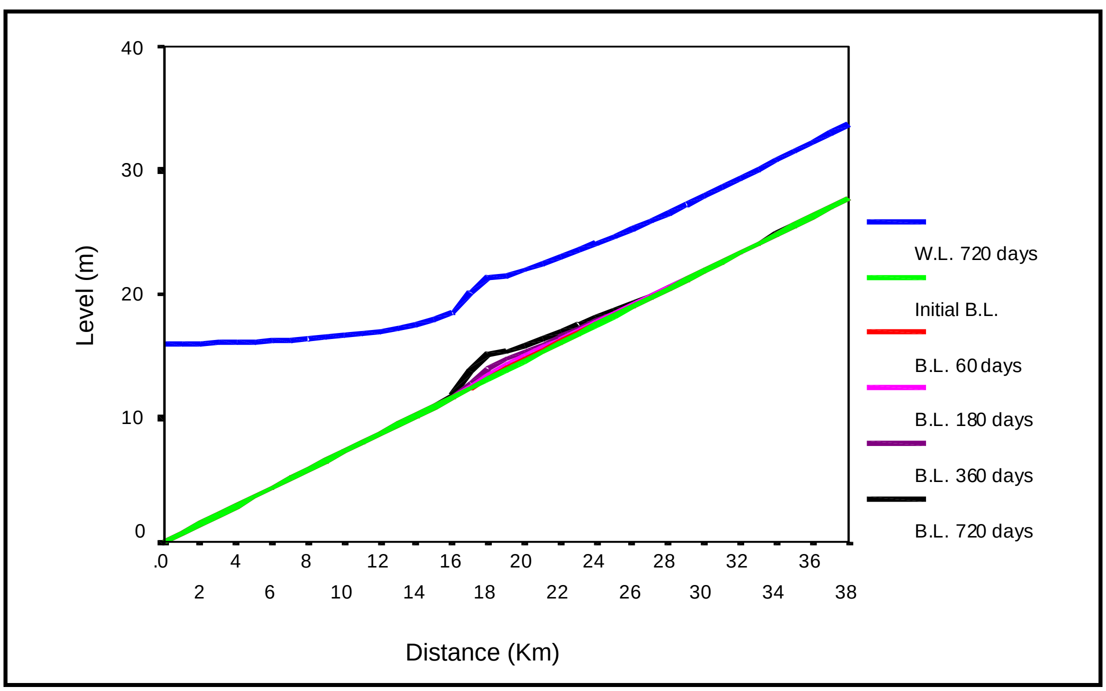

Fig. (5.2.4) Simulation of Bed Evolution (Shear Stress Parameter=0.16)

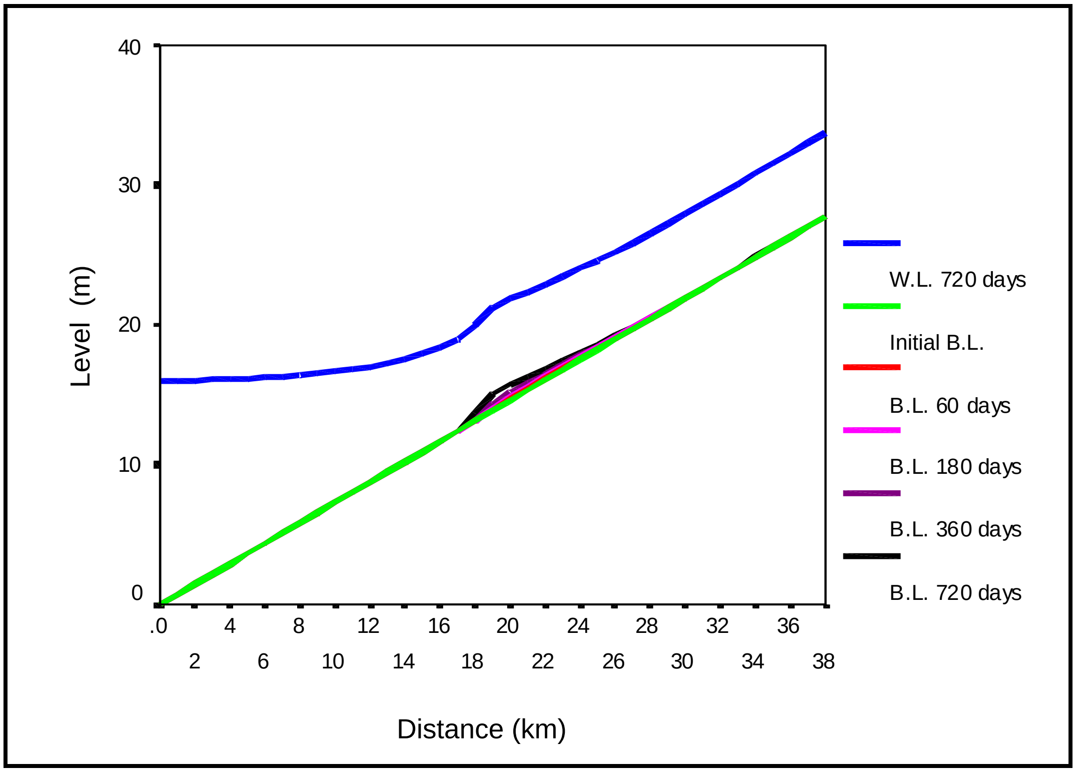

Fig. (9.4.5) Simulation of Bed Evolution (Shear Stress Parameter=0.18)

5-2-2 SIMULATION OF SEDIMENT TRANSPORT RATE

As the volume of the accumulated sediment is considered an important measure in the simulation of the delta formation, the sediment transport rate should be computed using the suitable sediment discharge predictor. The sediment transport rate per unit channel width, along the Rhone River reach has been computed applying the newly proposed bed-load predictor, equation (3.4.1.2) and using the values of the dimensionless shear stress parameter stated before. The effect of variation of the dimensionless shear stress parameter using the values 0.04, 0.06, 0.16 and 0.18 are shown in figure (5.2.6) figure (5.2.7), figure (5.2.8) and figure (5.2.9) respectively. These figures show the longitudinal profile of the sediment transport rate per unit width of the channel at different simulation times, 60 days, 180 days, 360 days and 720 days. The same value of the sediment coefficient 0.03 was used in the model to simulate the sediment transport rates.

The profiles shown correspond to various values of the dimensionless shear stress parameter. It is noticed that, as the shear stress parameter be smaller, the longitudinal profile of the sediment transport rate varies irregularly with the variation in simulation time. On the other hand, as the shear stress parameter be larger, the longitudinal profile of the sediment transport rate become more uniform with the variation in simulation time. It is clear that the sediment transport rate increases as the dimensionless shear stress parameter decreases, while the transport rate decreases as the parameter value increases. Furthermore, it is noticed that there is no sediment transport immediately upstream the dam for distance where the delta configuration is formed. This distance is shown to be longer as the shear stress parameter become smaller.

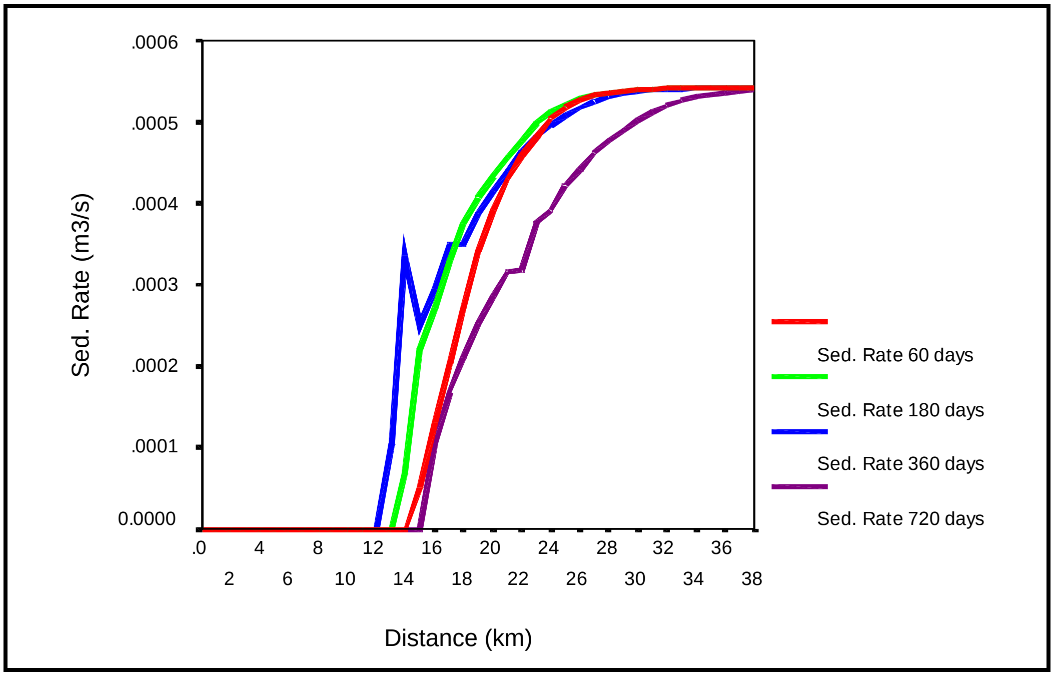

Fig. (5.2.6) Sediment Transport Longitudinal Profile (Shear Stress Parameter=0.04)

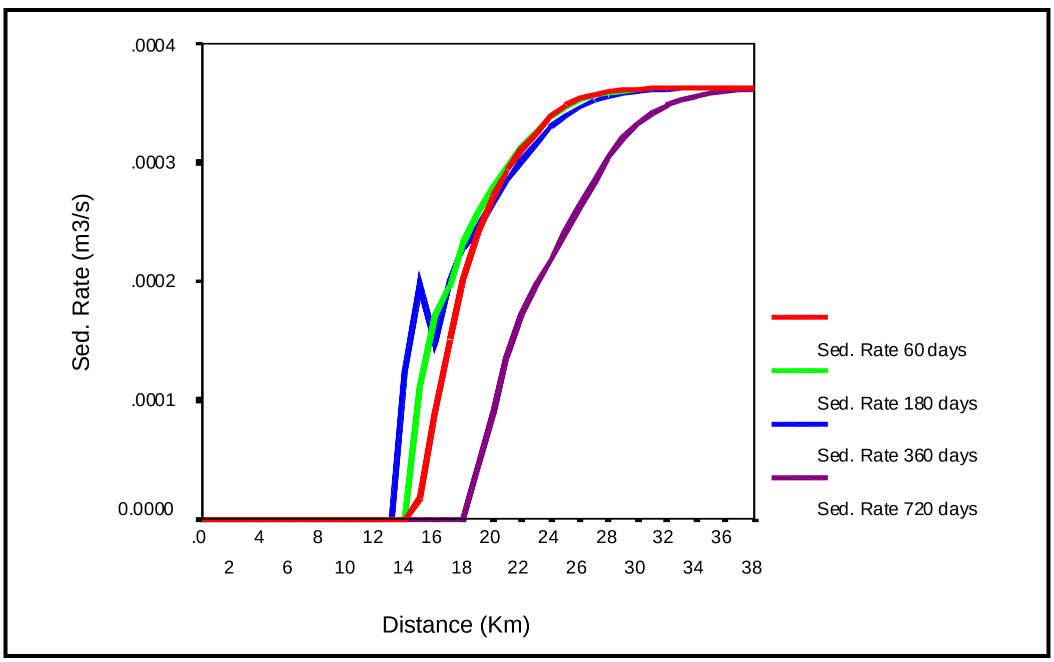

Fig. (5.2.7) Sediment Transport Longitudinal Profile (Shear Stress Parameter=0.06)

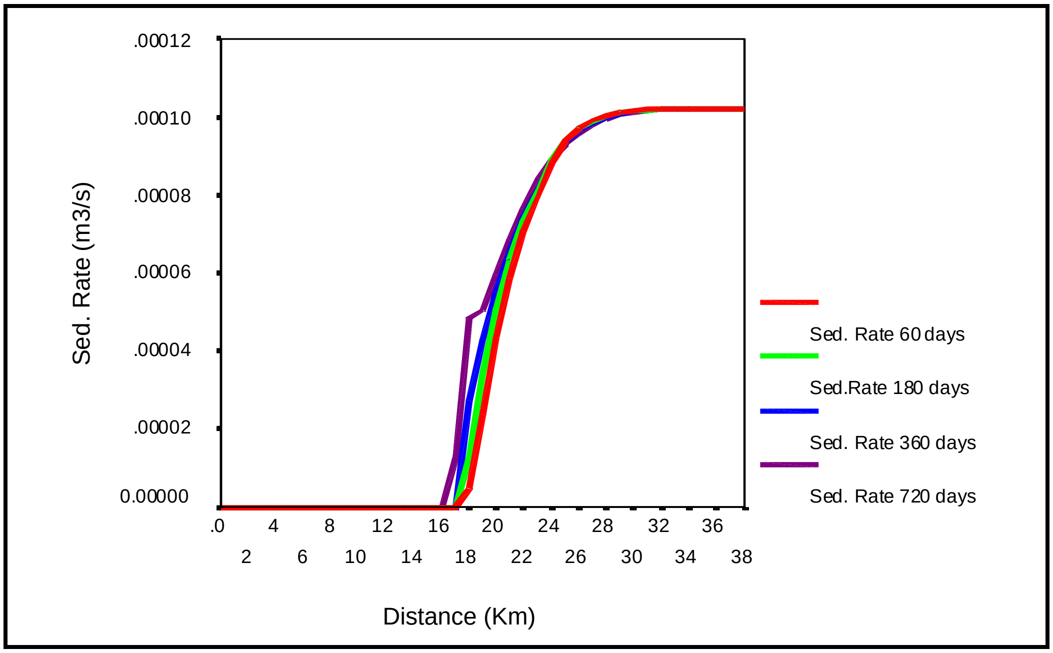

Fig. (5.2.8) Sediment Transport Longitudinal Profile (Shear Stress Parameter=0.16)

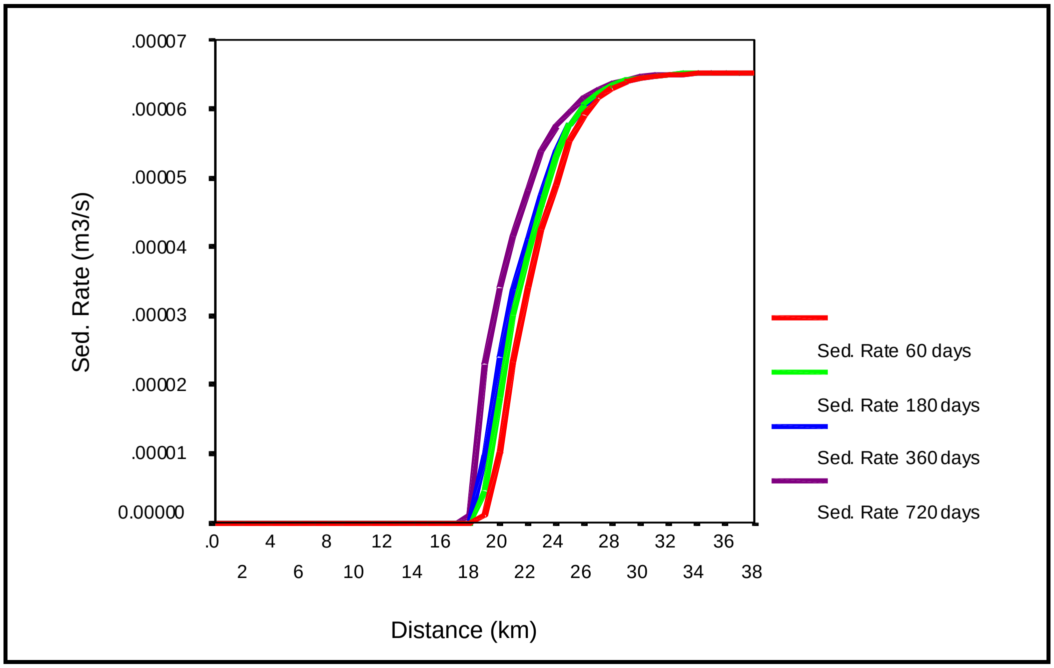

Fig. (5.2.9) Sediment Transport Longitudinal Profile (Shear Stress Parameter=0.18)

5-2-3 SIMULATION OF TEMPORAL EVOLUTION

Temporal evolution of bed level of the simulated reach was computed for different sections along the channel. The sediment transport rate per unit width of the channel at each section was computed using the same range of values for the dimensionless shear stress parameter. Figure (5.2.10), figure (5.2.11) and figure (5.2.12) represent the temporal simulation of the bed changes corresponding to the shear stress parameter values of 0.04, 0.12 and 0.18 respectively. The temporal simulations shown are carried for sections 13, 15, 18, 20, 22 kilometer distant upstream the dam.

It is shown that when the shear stress parameter is small, the simulated sections have irregular shapes. While the simulated sections become more uniform as the parameter value be larger. Also, it is noticed, as mentioned previously, the sediment transport rate increases as the dimensionless parameter value decreases.

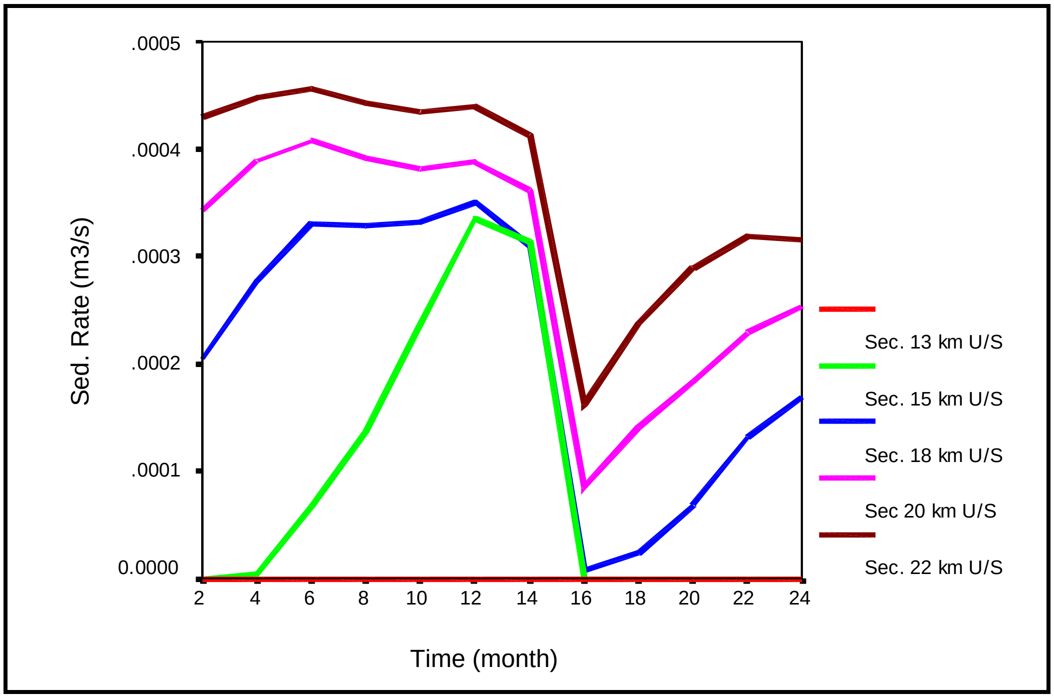

Fig. (5.2.10) Temporal Evolution of Sediment Rates (Shear Stress Parameter=0.04)

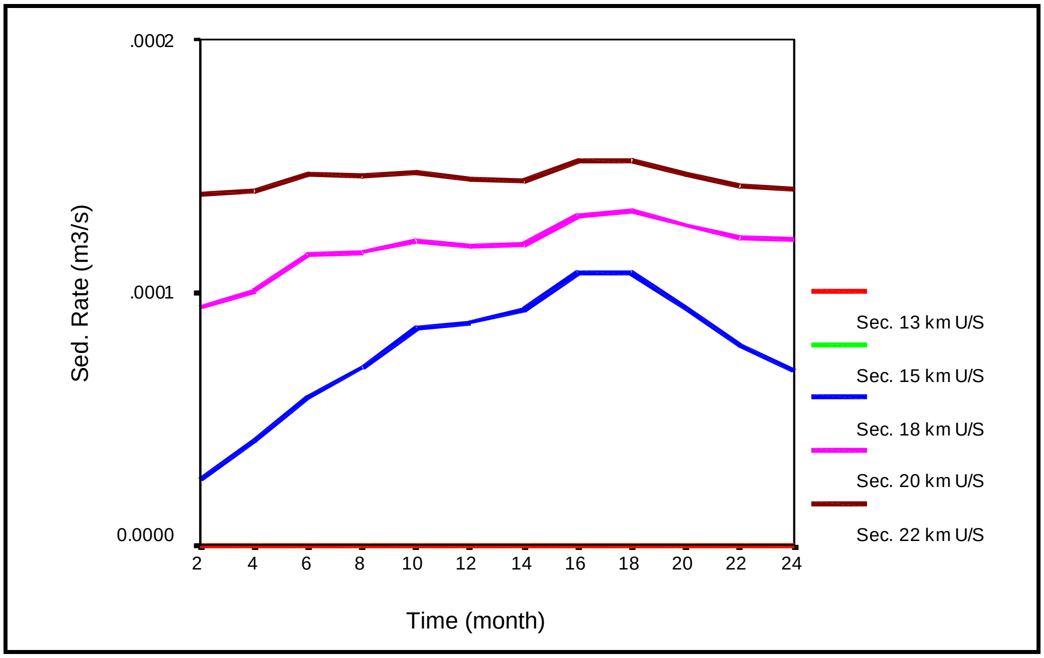

Fig. (5.2.11) Temporal Evolution of Sediment Rates (Shear Stress Parameter=0.12)

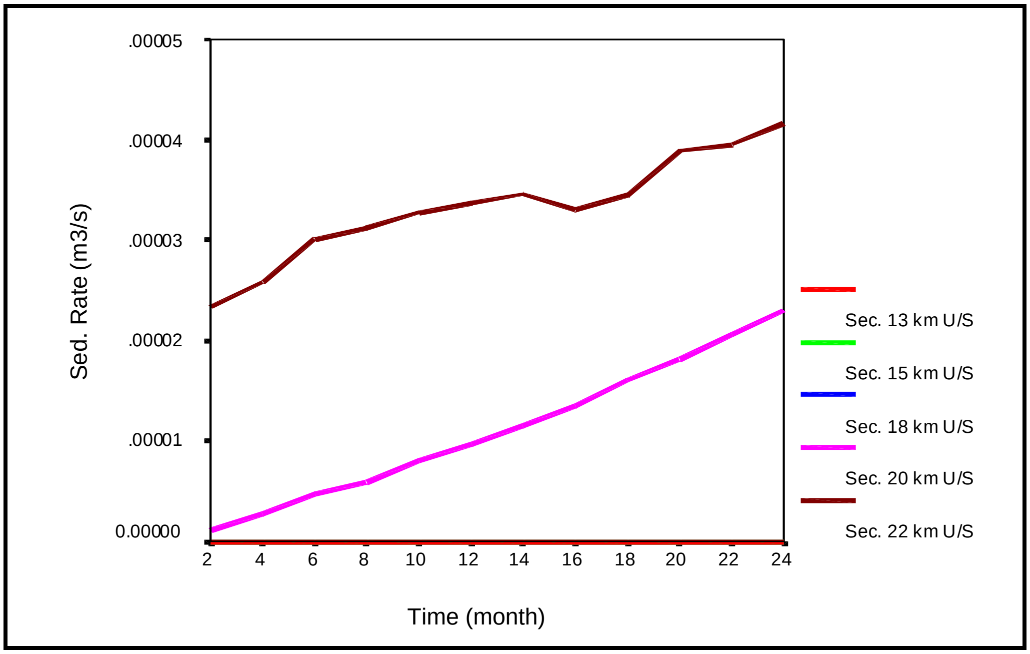

Fig. (5.2.12) Temporal Evolution of Sediment Rates (Shear Stress Parameter=0.18)

5-3 SEDIMENT COFFICIENT

In the same manner, as the effect of the dimensionless shear stress parameter on the formation of the delta shape and the sediment transport rate has been investigated. Now, the effect of varying the sediment coefficient values is presented. For the considered case of the Rhone River, two values of sediment coefficient have been tried in SEDTREN model. The shape of the delta formation corresponding to the dimensionless shear stress parameter of 0.18 and the selected two values of sediment coefficient 0.05 and 0.015 were chosen arbitrarily. The simulation results are presented in figure (5.3.1) and figure (5.3.2) respectively. It is shown that as the sediment coefficient value increased the delta formation shifted downstream according to its position when using a value of 0.03. Whereas, the delta formed upstream when a smaller sediment coefficient is used. The volume of the deposited sediment remains constant in all cases.

The values of 0.05 and 0.015 for the sediment coefficient were used again to simulate the sediment transport rate along the channel reach. Figure (5.3.3) and figure (5.3.4) shows the sediment transport longitudinal profile corresponding to the sediment coefficients 0.05 and 0.015 respectively. It is noticed that the sediment transport rate remains constant in both cases but there is a shift of the position of the beginning of the delta formation. The shift is downstream when the sediment coefficient is greater than 0.03 and upstream for smaller than 0.03.

The temporal bed evolution of the Rhone channel reach was also studied considering the variation in the sediment coefficient values. As mentioned in the above sections, the simulated delta is formed upstream as the sediment coefficient decreases. Figure (5.3.5) and figure (5.3.6) show the temporal evolution of the sediment rate at the selected sections using sediment coefficient values 0.05 and 0.015 respectively.

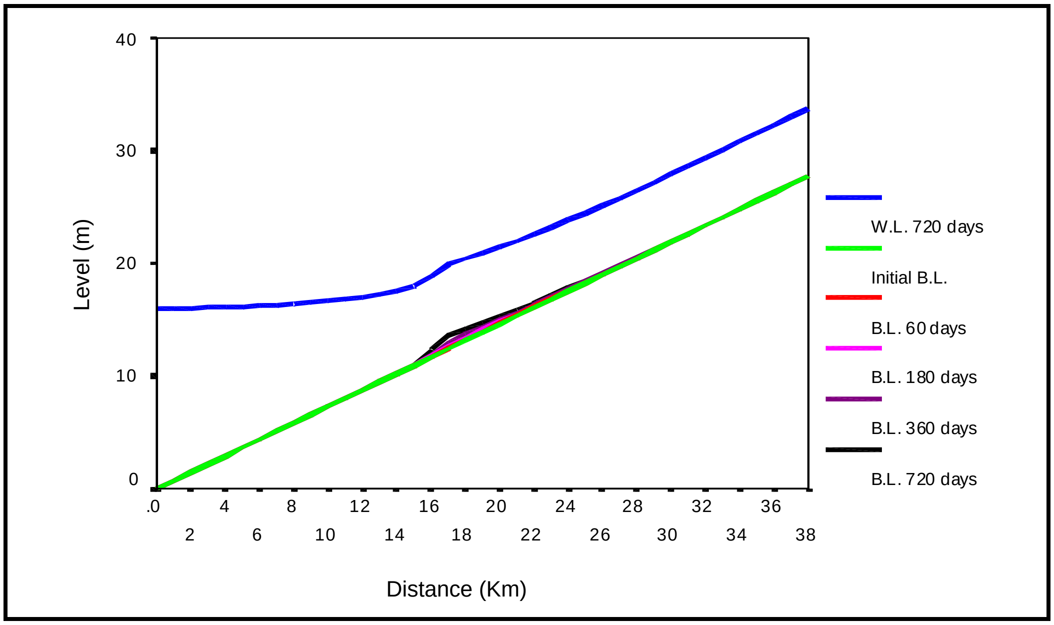

Fig. (5.3.1) Simulation of Bed evolution (Sediment Coefficient=0.05)

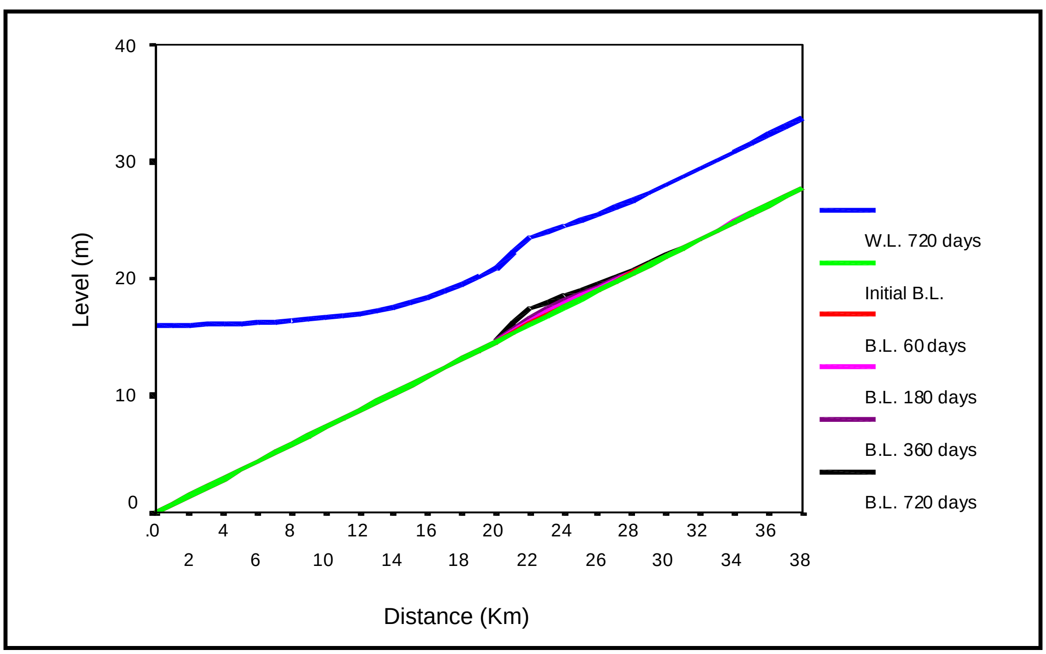

Fig. (5.3.2) Simulation of Bed evolution (Sediment Coefficient=0.015)

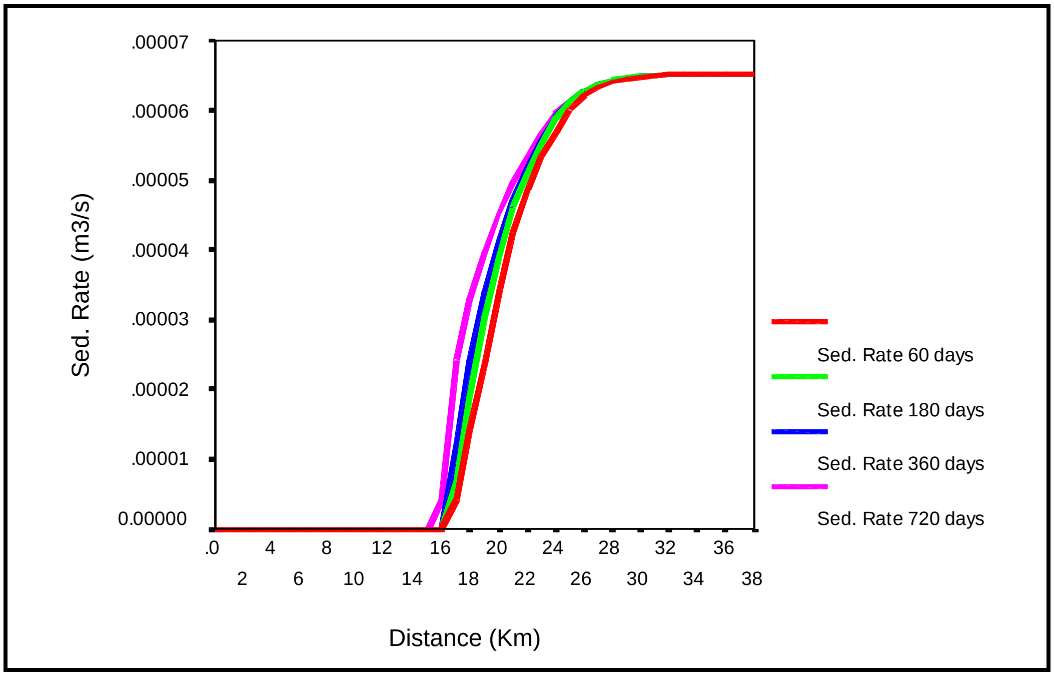

Fig. (5.3.3) Sediment Transport Longitudinal Profile (Sediment Coefficient=0.05)

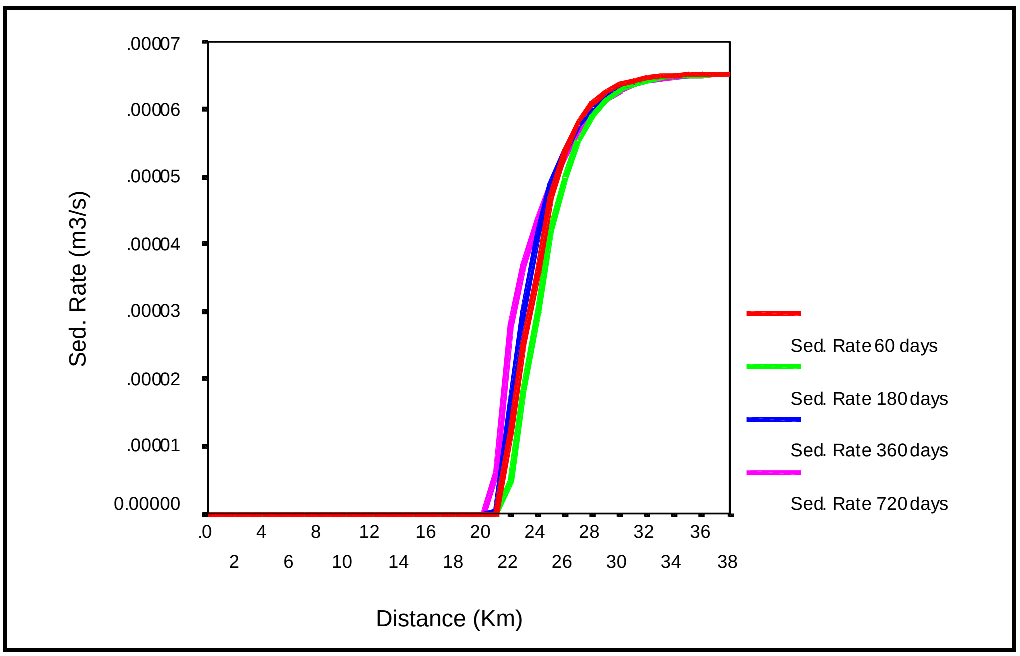

Fig. (5.3.4) Sediment Transport Longitudinal Profile (Sediment Coefficient=0.015)

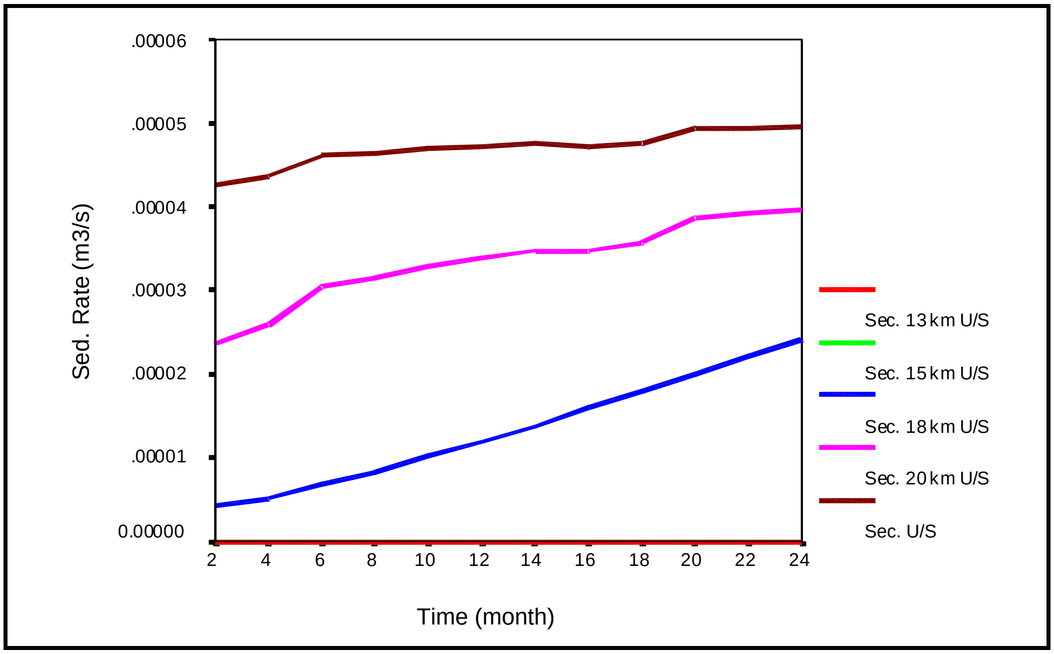

Fig. (5.3.5) Temporal Evolution of Sediment Rates (Sediment Coefficient=0.05)

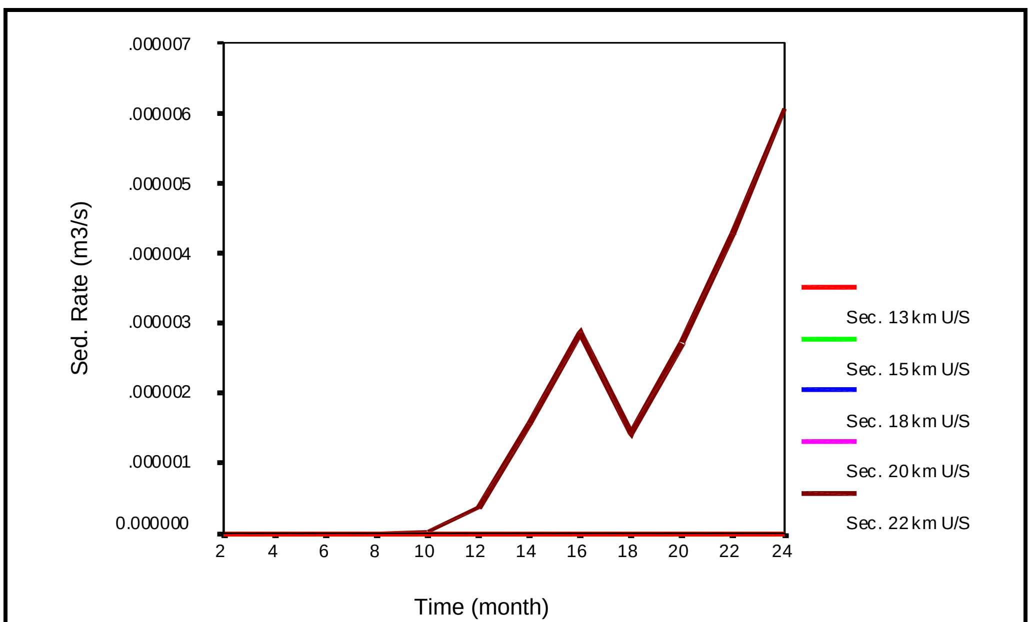

Fig. (5.3.6) Temporal Evolution of Sediment Rates (Sediment Coefficient=0.015)

5-4 TRANSPORTING ENERGY LINE

Previously, it was shown that, part of the energy head is consumed in transporting the sediment particles in alluvial channel bed and represented by equation (3.3.2.7). In a case of a reservoir pool upstream of a dam, the sediment transporting energy will increase starting from a point upstream the uniform flow depth towards the downstream direction. Thus, such increase in water depth will cause the sediment motion to be ceased at a certain section upstream the barrier. Accordingly, at that section the transporting energy line will decrease until the energy reverse back. Downstream that section the water depth will resist the motion of the sediment layer in the channel bed. Thus, this situation of the transporting energy may explain the phenomenon of the formation of the transported bed load in a delta shape in the reservoir at the farthest upstream, leaving the suspended sediment to settle down in the static water pool immediately upstream the dam.

The transporting energy, as newly developed concept in this research, was plotted in addition to the energy lines usually drown. Figure (5.4.1) represents the longitudinal profile of the channel reach of the Rhone River. The plotted lines elaborate the total energy line and the transporting energy line, from which it is shown that the energy dissipated in transporting the sediment particles increases as going downstream until reach a point where the potential energy head become dominant. At that position the transporting energy head diminishes. The dimensionless shear stress parameter value is 0.18 and the sediment coefficient value is 0.03. Lastly, investigation of the effect of the sediment coefficient is elaborated by plotting the transporting the energy line. Figure (5.4.2) and figure (5.4.3) represent the longitudinal profile of the channel reach corresponding to the coefficient values 0.05 and 0.015 respectively.

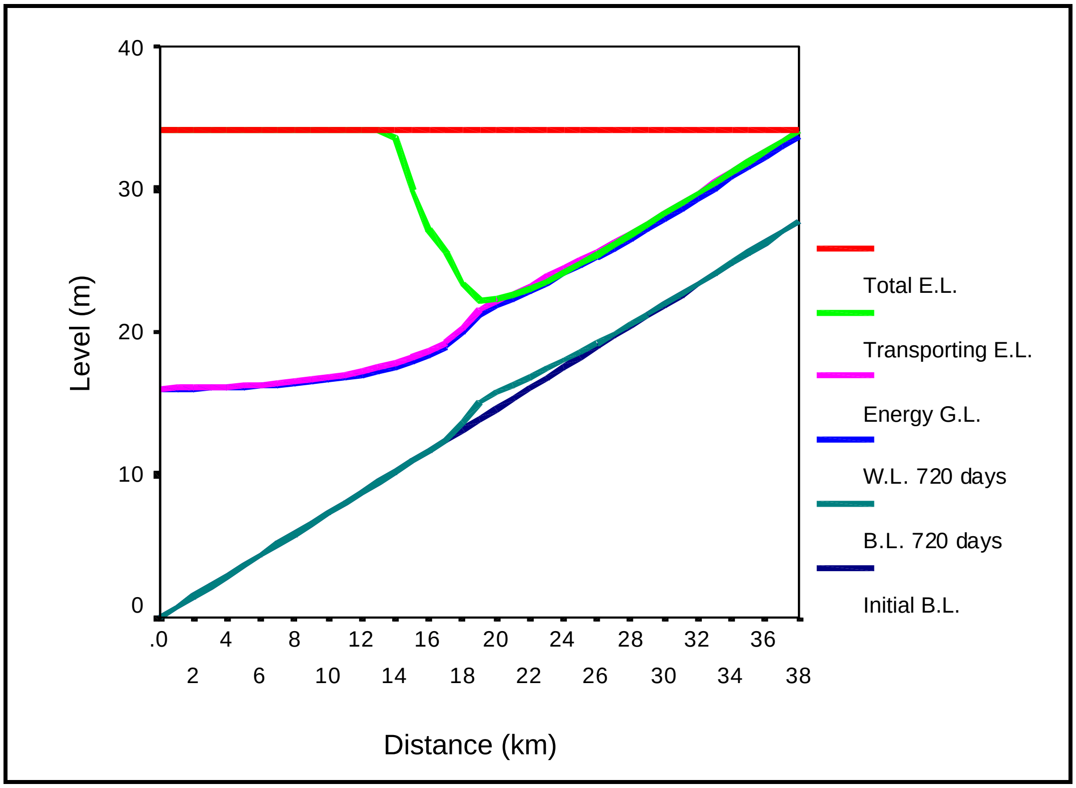

Fig. (5.4.1) Longitudinal Profile (Sediment Coefficient=0.03)

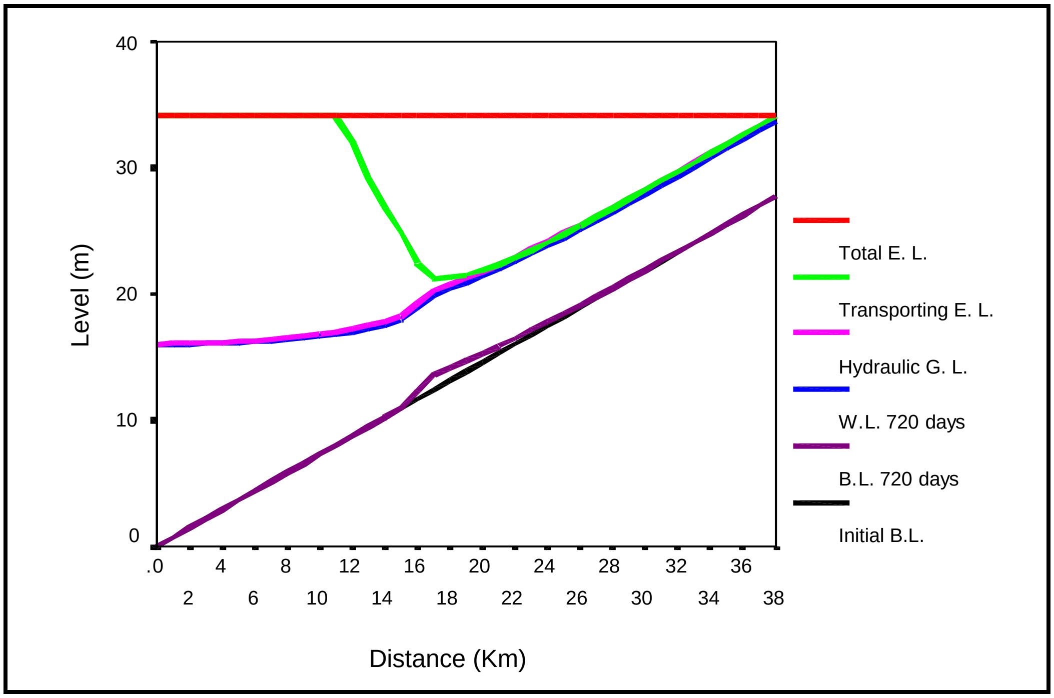

Fig. (5.4.2) Longitudinal Profile (Sediment Coefficient=0.05)

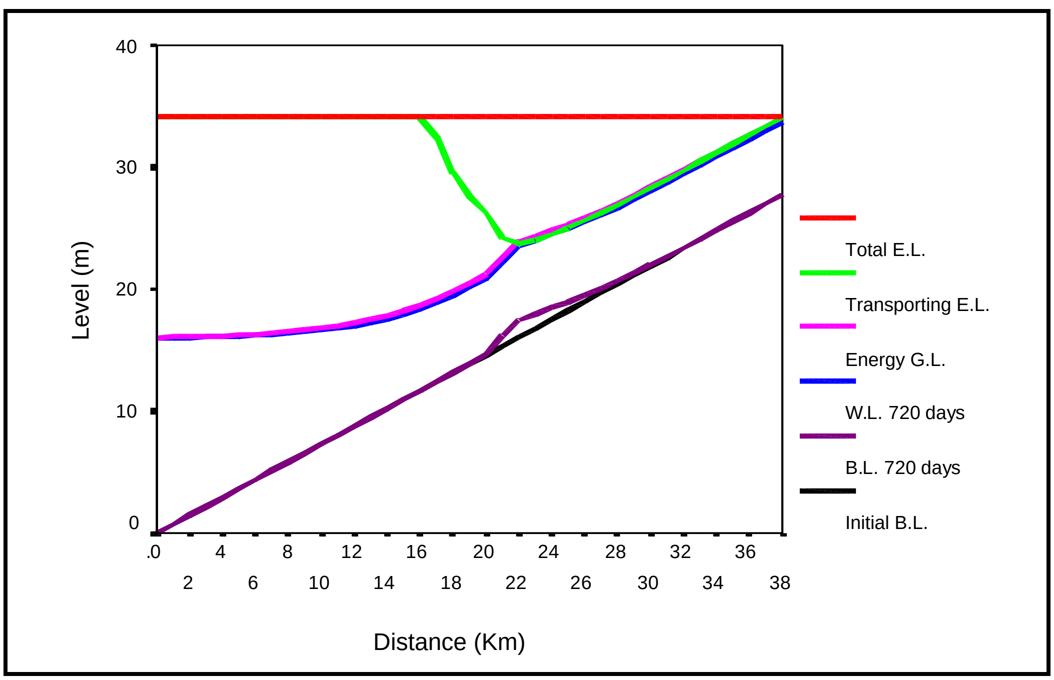

Fig. (5.4.3) Longitudinal Profile (Sediment Coefficient=0.015)

5-5 COMPARISON BETWEEN SEDTREN AND CARICHAR MODELS

Comparison of SEDTREN model results, the formation of the delta shape sediment deposition, using the three sediment coefficient values, 0.05, 0.015 and 0.03, with the result of Rahuel's CARICHAR model using Bell and Sutherland loading law, shows that the sediment coefficient should be larger so the delta is formed further downstream. Running SEDTREN model using sediment coefficient ranging between 0.015 and 0.15, a coefficient value 0.085 was found to simulate the delta formation in the same position as given by CARICHAR simulation in figure (4.3.2).

Table (5.5.1) shows the computations of the SEDTREN model. Figure (5.5.1) translates these computations into a graphical representation. The simulation of the sediment transport by SEDTREN model was found to be, to some extent, close to the sediment rate computed by CARICHAR model. Figure (5.5.2) shows the SEDTREN simulation of the sediment transport rate along the channel reach compared to the CARICHAR's in figure (4.3.4) and figure (4.3.5) shown previously.

Table (5.5.1) SEDTREN Model Computations (Sediment coefficient=0.085)

Sec.	T.E.L. (m)	Tr.E.L. (m)	E.G.L. (m)	W.L. (m)	Initial B.L. (m)	Final B.L. (m)
0	34.1067	34.1067	16.05097	16	0	0
1	34.1067	34.1067	16.07912	16.02333	.73	.73
2	34.1067	34.1067	16.11185	16.05056	1.46	1.46
3	34.1067	34.1067	16.15016	16.08255	2.19	2.19
4	34.1067	34.1067	16.19527	16.12039	2.92	2.92
5	34.1067	34.1067	16.24877	16.16547	3.65	3.65
6	34.1067	34.1067	16.31827	16.22528	4.38	4.38
7	34.1067	34.1067	16.40211	16.29787	5.11	5.11
8	34.1067	32.36995	16.50393	16.38662	5.84	5.84
9	34.1067	30.34419	16.62832	16.49589	6.57	6.57
10	34.1067	28.43396	16.78103	16.63118	7.3	7.3
11	34.1067	26.66947	16.96897	16.79929	8.030001	8.030001
12	34.1067	24.94699	17.21676	17.02351	8.760001	8.806635
13	34.1067	22.27586	17.87708	17.64204	9.490002	10.19127
14	34.1067	20.7126	18.6202	18.34884	10.22	11.41463
15	34.1067	20.57174	18.95944	18.67205	10.95	11.934
16	34.1067	20.52925	19.35937	19.05654	11.68	12.49249
17	34.1067	20.61307	19.81665	19.49993	12.41	13.08143
18	34.1067	20.83298	20.32114	19.99277	13.14	13.68918
19	34.1067	21.17547	20.86623	20.52857	13.87	14.31226
20	34.1067	21.62142	21.442	21.09728	14.60001	14.94496
21	34.1067	22.14645	22.04825	21.6983	15.33001	15.59214
22	34.1067	22.73068	22.6863	22.33255	16.06001	16.25923
23	34.1067	23.35751	23.36798	23.01151	16.79	16.96143
24	34.1067	24.00052	24.05035	23.69156	17.52	17.66107
25	34.1067	24.68221	24.72065	24.36056	18.25	18.34096
26	34.1067	25.38358	25.41153	25.05067	18.98	19.03751
27	34.1067	26.09714	26.11687	25.75557	19.71	19.74611
28	34.1067	26.81791	26.83125	26.4697	20.44	20.46232
29	34.1067	27.5428	27.55171	27.19002	21.17	21.1838
30	34.1067	28.27001	28.2759	27.91413	21.9	21.90854
31	34.1067	28.99848	29.00235	28.64054	22.63	22.63528
32	34.1067	29.72766	29.7303	29.36847	23.36	23.3634
33	34.1067	30.45721	30.45894	30.09709	24.09	24.09213
34	34.1067	31.18697	31.18814	30.82629	24.82	24.82137
35	34.1067	31.91684	31.91749	31.55564	25.55	25.55075
36	34.1067	32.64677	32.64741	32.28555	26.28	26.28069
37	34.1067	33.37673	33.37671	33.01486	27.01	27.01
38	34.1067	34.1067	34.1067	33.74485	27.74	27.74

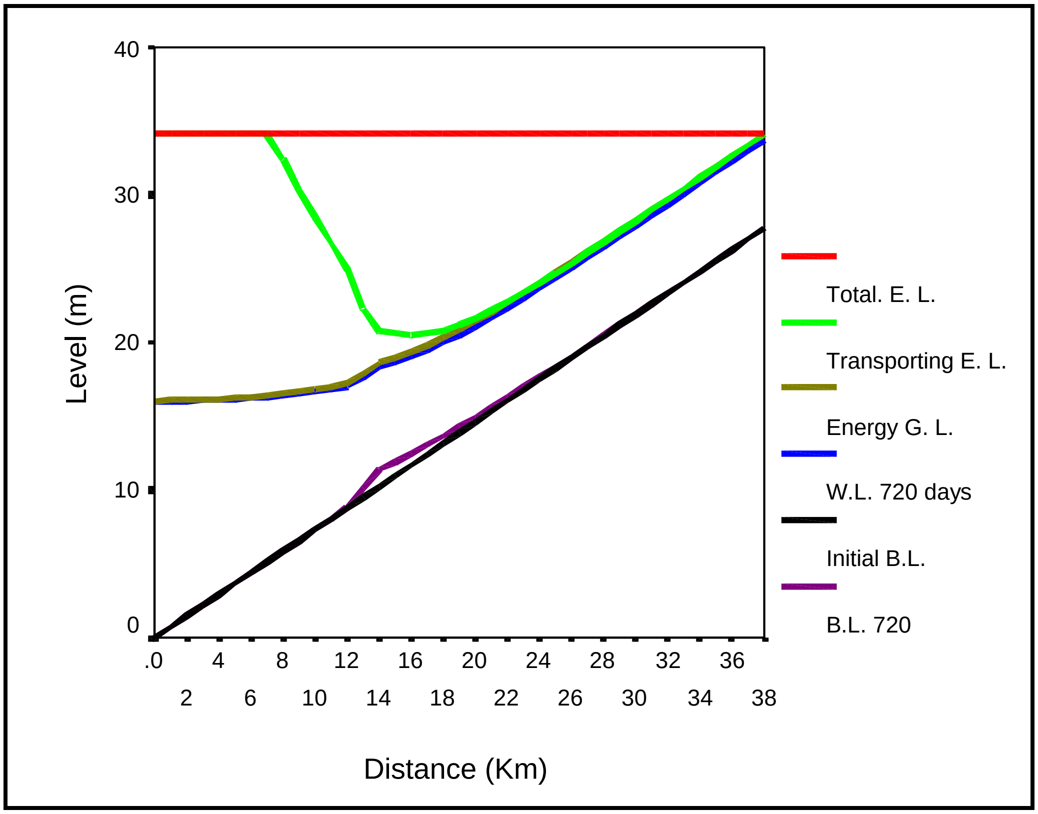

Fig. (5.5.1) Longitudinal Profile (Sediment Coefficient=0.085)

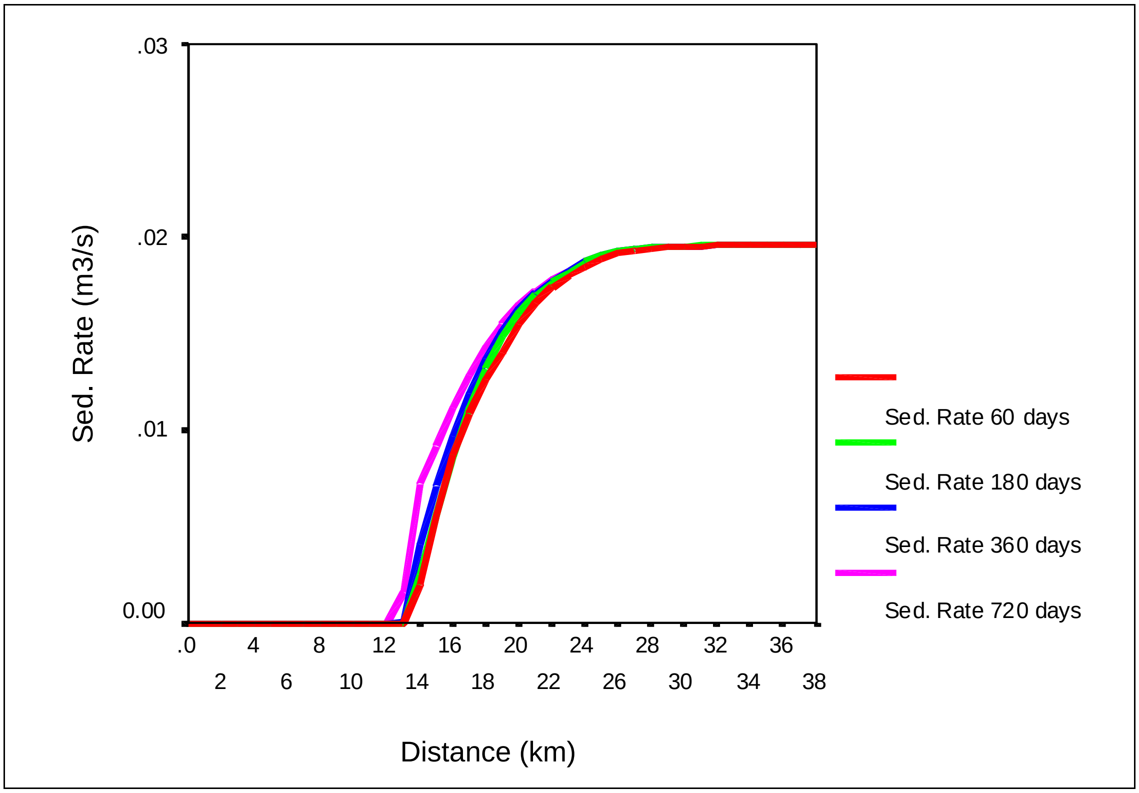

Fig. (5.5.2) Sediment Transport Longitudinal Profile (Sediment Coefficient=0.085)

5-6 SIMULATION OF ACTIVE LAYER

The active layer as the transport medium of the sediment particles was simulated considering the armor diameter and the percentage size distribution. The diameter of the immobile bed sediment particles was computed at each time step. Figure (5.6.1) shows the longitudinal profile of the Rhone River reach representing the armor diameter at the end of simulation period. Sediment particles size distribution is also computed at the end of each computational time step. Figure (5.6.2) elaborates the situation of particles distribution in the active layer. It is noticed the percentage distribution in the position of the delta formation is clearly changed for the three classes of the considered sediment mixture. A mobility coefficient value 0.1 was used in the model in order to count for the exchange of sediment particles in the active layer.

5-7 COMPUTATION OF FLOW CHARACTERISTICS

The flow characteristics were computed using SEDTREN model. This includes the previously mentioned dimensionless numbers. Figure (5.7.1) represents the longitudinal profile of the channel showing Froude number. It is clearly noticed that the flow is sub-critical along the simulated reach. On other hand, the computation of the sediment particles Reynolds number for the three sediment classes is represented in figure (5.7.2). The situation of the channel is clearly shown, the reach downstream the delta is not subjected to sediment transporting energy so that the dimensionless particles Reynolds number zero.

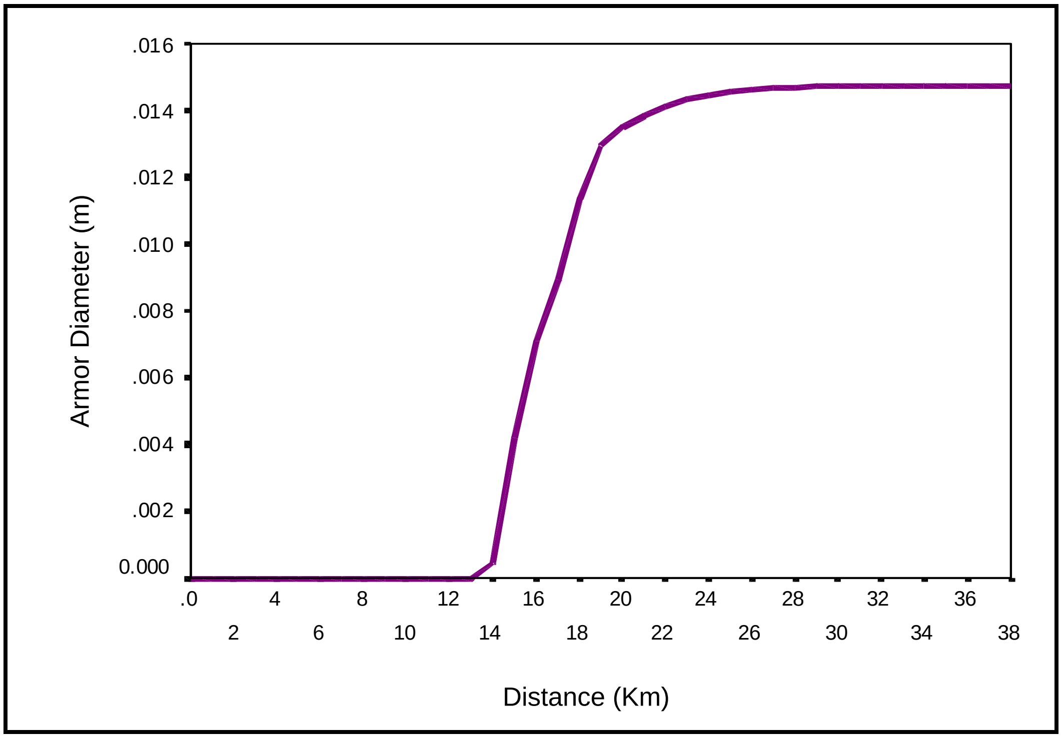

Fig. (5.6.1) Longitudinal Profile for armored diameter.

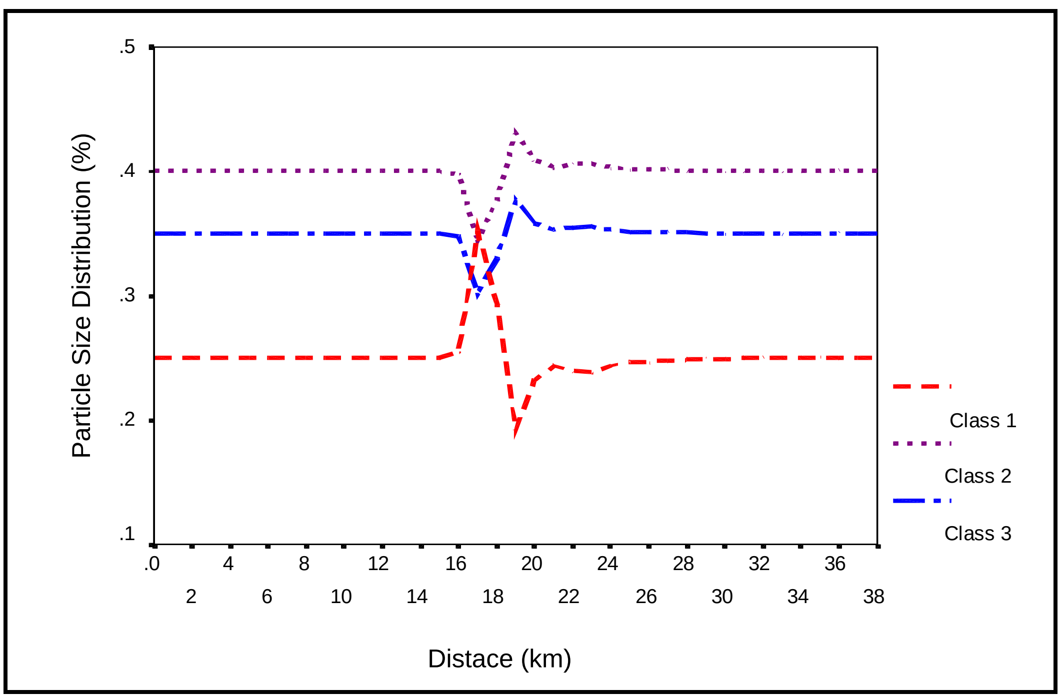

Fig. (5.6.2) Longitudinal Profile for Particle Size Distribution.

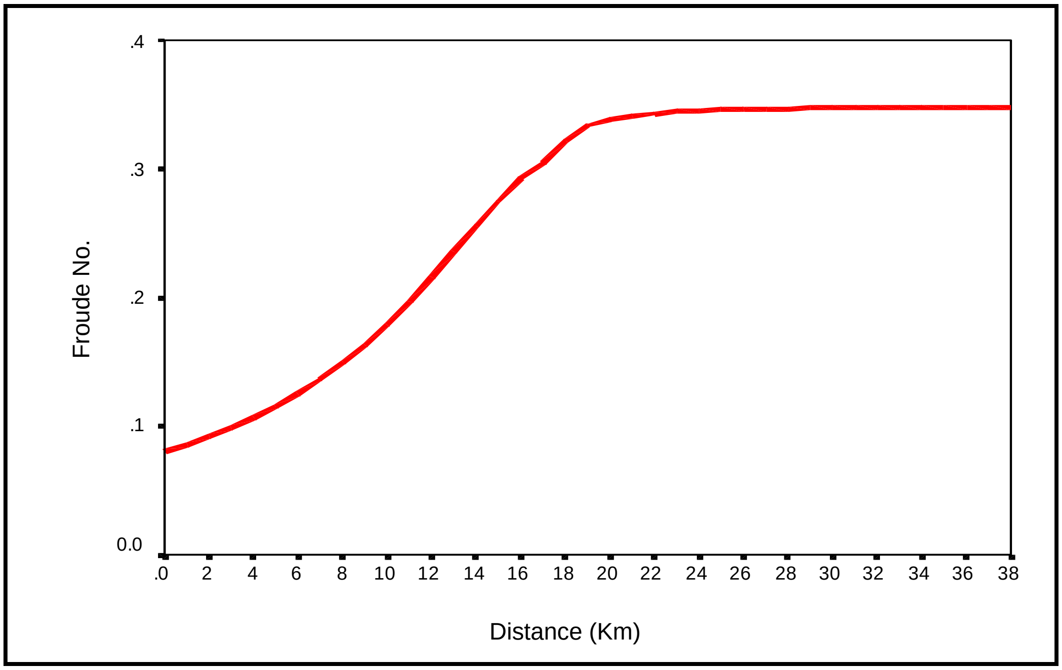

Fig. (5.7.1) Longitudinal Profile for Froude No.

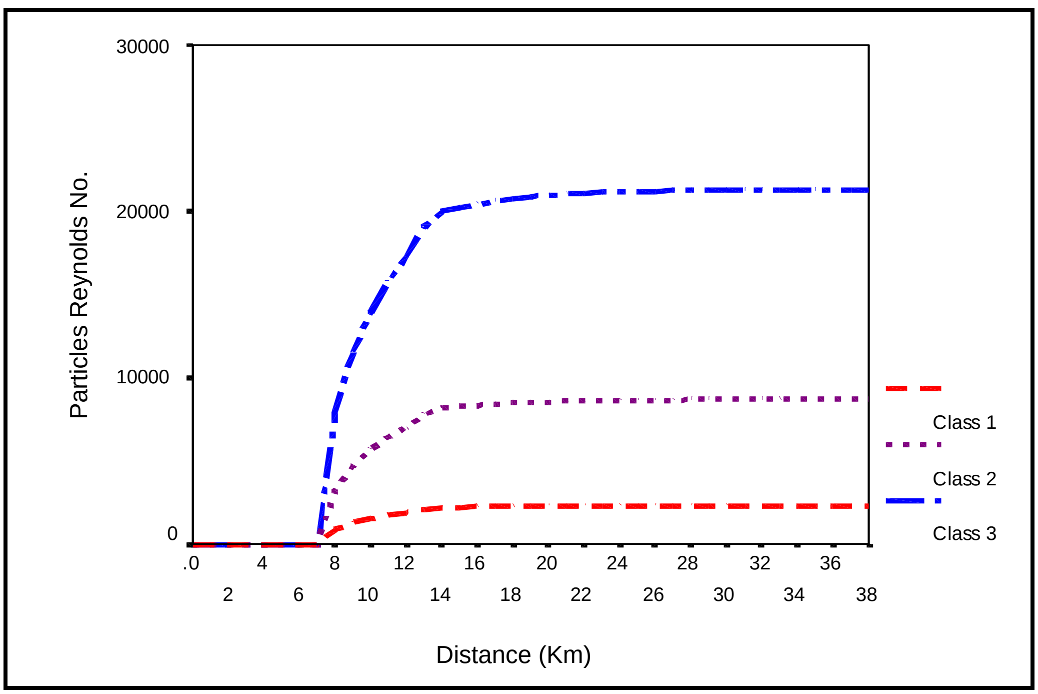

Fig. (5.7.2) Longitudinal Profile for Particles Reynolds No.

5-8 MODEL LIMITATIONS

In order to specify the model limitations, another alluvial reach, Atbara River, was simulated using the model. Atbara River is a seasonal river that originates from the Ethiopian Plateau to join the River Nile at Atbara in Sudan. Its annual discharge varies from 6 milliard m^3 in a low flood year to about 24 milliard m^3 in a high flood year. The average slope of river is 46 cm/Km. Khashm El Girba Dam was constructed, in 1964, on the river with a design capacity of 1.2 milliards m^3 in a reservoir extends for 84 Km length and max pool level 40m, Hussein (1994). The bed load is estimated as 10-15 % of the total accumulated sediment, Mohamed et al (2001). Since 1971 flushing of sediment during the flood season is being practiced, Osman (2002).

The bed material is composed of silt, sand and fine gravel. The model was applied using a dimensionless shear stress parameter value 0.18 and a sediment coefficient value of 0.085, which used for Rhone River reach. The simulated sediment accumulation was found to be 32 million m^3 about 10% of total sediment accumulated during the first five years of the dam commissioning. Figure (5.8.1) represents the simulation of bed evolution during a period of five years. The profile of the sediment transport rate is shown in figure (5.8.2).

It is clearly noticed from the two simulated cases that the proposed model, with dimensionless shear stress parameter 0.18, simulates the bed load of larger particle sizes, Rhone River, in a good agreement with other models. On the other hand, simulation of Atbara River with smaller bed material particle sizes, showed a lower agreement. This is noticeable in simulating the sediment rate profile, where oscillations occurred. Using larger values of shear stress parameter showed better results for the simulated Atbara

reach. Figure (5.8.3) shows the bed evolution of Atbara using shear stress parameter value 0.16 and sediment coefficient value 0.085. The sediment transport rate is shown in figure (5.8.4). It is recommended that more investigation and analysis to carried on such rivers with smaller bed size.

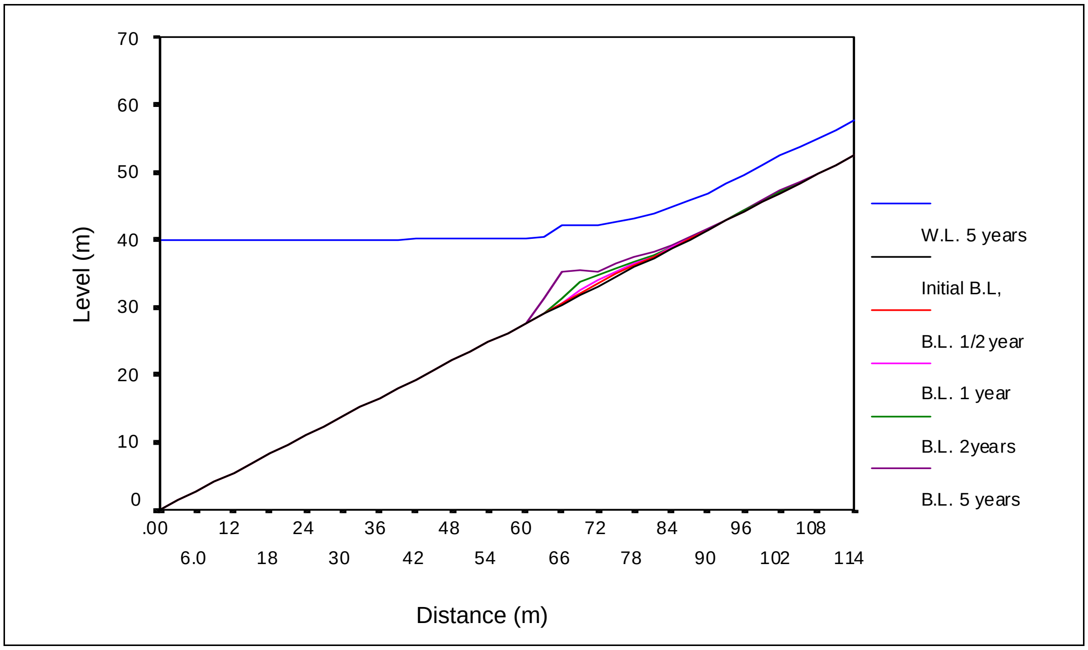

Fig (5.8.1) Simulation of Bed Evolution Atbara River (Dimensionless Shear Stress Parameter 0.18)

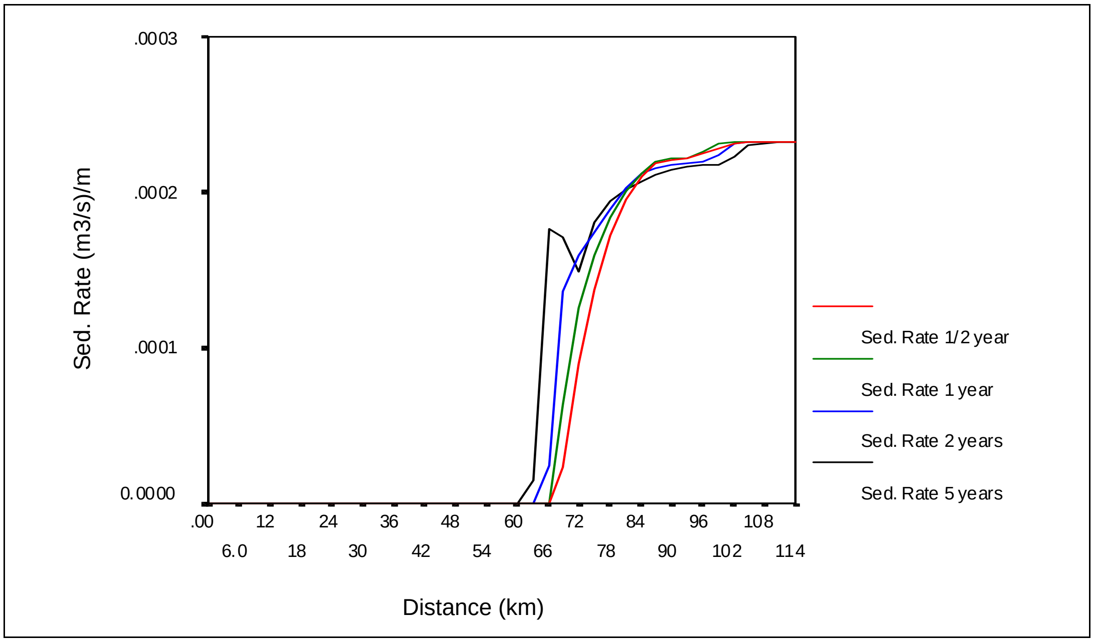

Fig. (5.8.2) Longitudinal Profile of Sediment Transport Atbara River

(Dimensionless Shear Stress Parameter 0.18)

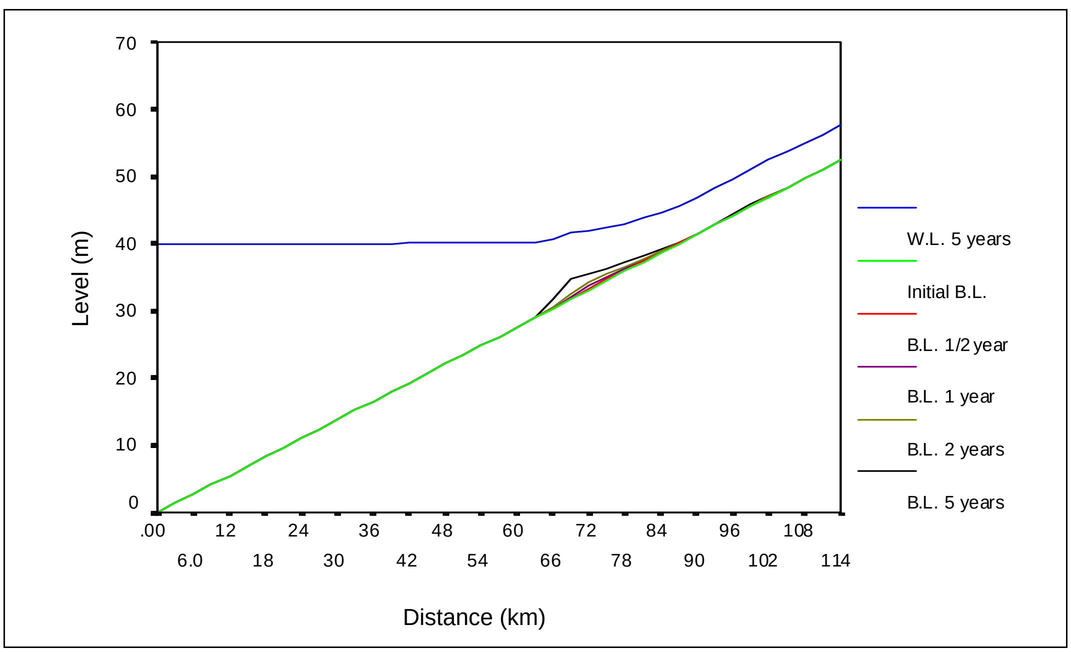

Fig. (5.8.3) Simulation of Bed Evolution Atbara River

(Dimensionless Shear Stress Parameter 0.16)

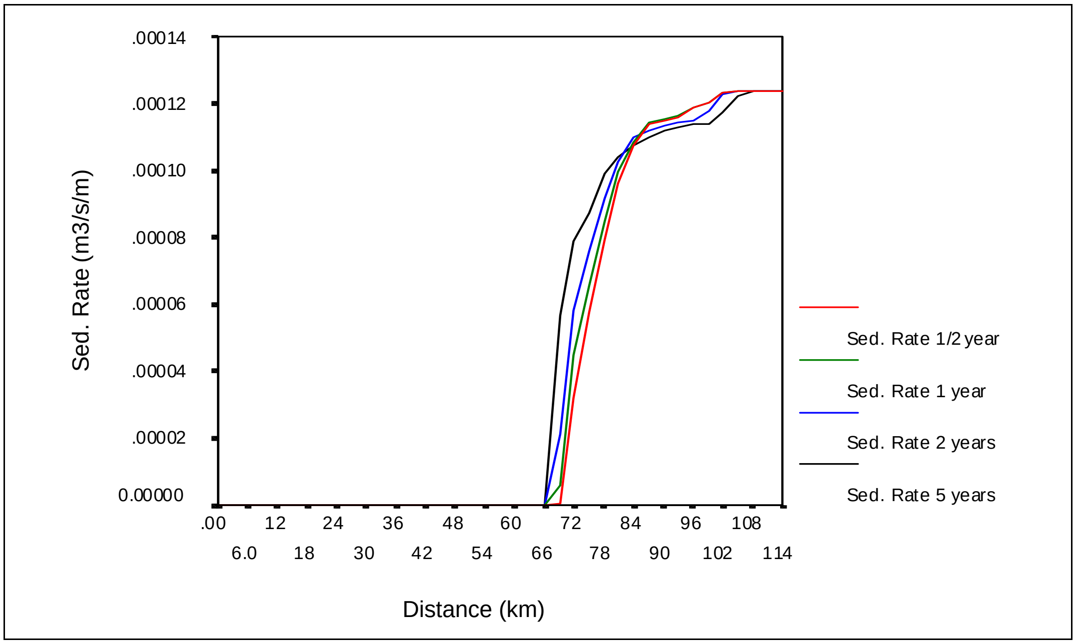

Fig. (5.8.4) Longitudinal Profile of Sediment Transport Atbara River (Dimensionless Shear Stress Parameter 0.16)

CHAPTER SIX

CONCLUSION AND RECOMMENDATIONS

6-1 CONCLUSION

Long-term river- bed evolution due to sediment transport became a significant part in river flow modeling. Water-sediment routing models, to simulate such changes, have been commonly developed. Furthermore, this technique directed the attention for the need to new concepts and developments. In particular, the governing equations of the phenomenon and the complementary relations have to be thoroughly investigated. In this research, an attempt was carried to investigate the mathematical relations governing the phenomenon starting from the basic principles. The sediment transporting energy line was considered as a significant parameter in mobile-bed modeling. The computation of sediment transport rate was also investigated as a driving parameter. Additionally, the bed composition was introduced as another important parameter. Accordingly, a mobile-bed model was formulated considering the newly developed concepts.

As the equations related to water flow are essentially derived for fixed-bed channels, the slope of the steady state energy line is considered as an approximate function. Thus, the assumption stated by some researchers that, part of the energy is consumed in transporting the sediment particles in alluvial was investigated in order to determine the transporting energy head. Two approaches were followed to make such investigation. The first is a theoretical investigation through proposing a modified gradually varied flow equation, for alluvial channel systems, as a function of a sediment characteristic. The second approach is to study a conceptual sediment layer, transported on an alluvial channel bed, in order to determine the part of the energy head transporting it.

Theoretical development of the modified gradually varied flow equation to compute the flow surface profile in alluvial channels was presented elaborating the basic considerations and assumptions. The modification of the gradual flow equation, usually used for fixed-bed channels, was proposed through introducing a set of modification factors to correct the main parameters of the equation, namely, the normal flow depth, the critical flow depth and the water slope. As part of the energy of the flowing water is consumed in transporting the sediment particles, this implies that the friction slope will decrease. In addition, the specific energy will be reduced, and the water slope in alluvial channel is considered to reduce as a result of increase of the channel roughness. Consequently, the normal depth modification factor, the critical depth modification factor and the water slope modification factor were assumed to be less than unity as the normal flow depth, the critical flow depth and the water slope were reduced.

The proposed modification factors were investigated through numerical experimentation. Data available from one of the large-scale laboratory flume channels namely, St. Anntony Fall Laboratory SAFL, were used in the numerical experimentation. Parametric analysis was made though the numerical experiments on SAFL data in order to investigate the effect of each modification factor and to reach an optimum set of values of the modification factors. Finally, the theoretical development was ended with proposing a sediment characteristics parameter to be related to each modification factor in an exponential form. This reduced the modified equation in a form investigated also on a conceptual basis.

Application of the momentum principle to rigid bed channels was continued in order to investigate in depth the term of the energy loss due to external forces and to extend

the application of this basic principle for a conceptual sediment layer in an alluvial channel reach. Resisting energy concept, energy head loss due to external forces, was developed. Consequently, the gradually varied flow formula developed indicated that the change in flow depth, the flow surface profile, is dependent on the downstream depth, the upstream depth and the averaged depth. Application of the momentum principle on a bed material layer in an alluvial channel led to evaluate the energy head consumed in transporting the sediment layer between two adjacent sections. It was found as a function of a sediment coefficient, specific weight of both the fluid and sediment particles, change in flow depth and change of the bed level.

The newly developed transporting energy concept is a very important parameter in the basic principles of alluvial channel hydraulics, because the transporting energy represents part of the energy dissipated in the channel. Application of the energy equation taking into account the transporting energy head as a part of the energy head dissipated in the alluvial channel led to modify the gradually varied flow equation derived on a conceptual basis. In this approach, the sediment characteristic parameter was derived this time as a basic parameter, the matter that ensures the assumption of the existence of exponential relation between the modification factors and the sediment characteristic parameter.

Comparison of the theoretical and the conceptual formulations showed that the constants and the exponents of the assumed exponential relations could be directly determined. Substitution of these values indicated that the normal depth modification factor is a constant and independent of the proposed sediment characteristics

parameter, while both the critical depth modification factor and the water slope modification factor are functions of the sediment characteristics parameter. Both factors are functions of the specific gravity and the sediment coefficient. The sediment coefficient, most probably, assumed to be related to the angle of repose of the sediment particles and the size distribution. More investigations are required to test the assumed relations.

The SEDiment TRansporting ENergy concept was the basis of the newly developed SEDTREN model. The formulated model treats the case as quasi-steady flow. The channel is divided into sub-reaches and the computations for each one are performed for successive discrete time intervals. The water surface profile in the channel is predicted by solving the gradually varied flow equation. Standard step method is used in order to calculate the water depths in upstream sections, subject to a known water surface elevation at the downstream boundary. The transporting energy head along the reach are computed then the bed shear stresses are evaluated. The sediment transport rates are then computed using the newly proposed formula, which an access shear type formula and a function of the transporting energy head. The formula is also dependent on a dimensionless shear stress parameter.

The active layer was treated as it consists of a sediment mixture of several discrete size fractions. Each class is delineated by two limiting diameters and represented by the geometric mean of the limiting diameters. Exchange of sediment particles occurs between the active layer and the

moving bed layer was considered each time step in order to allow for any erosion or deposition of different size fractions of sediment particles. Thickness of the active layer assumed to be given as twice the representative diameter multiplied by the ratio of the bed shear to the critical shear. Computation of the sediment discharge exchange and the sediment distribution were accomplished using the sediment conservation for each size class.

Sediment continuity equation is applied in computing the bed evolution along the channel. Sediment discharge for individual size class is determined at each section of the mixing layer bed material. Exener equation of sediment continuity was discretized to calculate the depth eroded or deposited by the different sediment size fractions in each sub-reach. The armored diameter, the smallest sediment size that is immobile under the available shear stress, was computed in each section along the channel reach. The flow variables at each section, including Froude number and Reynolds number of the flow and the sediment particles Reynolds number, were computed in each computational time interval.

A schematic river reach having the overall hydraulic and sediment characteristics of the lower Rhone River, in France, was used by Rahuel et al (1989) in CARICHAR model. The available field data of that reach was used in this part of the research to calibrate the newly developed SEDTREN model comparing the results with those presented by Rahuel et al (1989). Parametric analysis for the simulated reach of the River by SEDTREN model was carried to study the effect each

of the model parameters, the dimensionless shear stress parameter and the sediment coefficient. Longitudinal profiles of Rhone River reach were simulated representing the rate of the transported bed load. In addition, the temporal evolution in various sections of the channel reach was carried for different values of the dimensionless shear stress parameter. The dimensionless shear stress parameter found to be related to the delta shape formation. In the same manner, effect of varying the sediment coefficient value was investigated. The sediment coefficient value found to be related the position of the delta formation along the channel reach.

The transporting energy, as newly developed concept in this research, was plotted in addition to the energy lines usually drown representing the longitudinal profile of the channel reach. The plotted lines elaborated the total energy line and the transporting energy line, from which it was noticed that the energy dissipated in transporting the sediment particles increases as going downstream until reach a point where the potential energy head became dominant, where the transporting energy head became zero. The shape of the transporting energy line seems to be an inverse shape to the delta formation on the channel bed.

Finally, A simplified simulation case was considered to apply SEDTREN model to the Atbara River reach. It was assumed that all suspended sediment would be retained in upstream reservoirs. Using arbitrary values of the dimensionless shear stress parameter and sediment coefficient, the sediment accumulation was modeled to

simulate 10% of total sediment accumulated during the first five years of the dam commissioning.

6-2 RECOMMENDATIONS FOR FURTHER WORK

The research benefited from the previous modeling techniques, great amount of sediment transport theories and the available laboratory and field data. To motivate additional future studies and investigations on the subject and the newly developed concepts, the following recommendations are suggested for further work:

1- The proposed modified gradually varied flow equation for alluvial channels need to be verified further by testing other field and laboratory data of different sediment properties in order to investigate the functional relation of the sediment characteristic parameter.

2- Newly developed gradually varied flow equation derived on the basis of the resisting energy head should be tested to ensure its validity by comparing it with the usual gradual flow equation.

3- Application of the momentum principle for a conceptual sediment layer in alluvial channel bed could be derived for a variable thickness in order to investigate the non-uniform transport of the sediment layer.

4- The general flow equation in alluvial channels derived on conceptual basis should be deeply investigated for various types of alluvial channel in order to develop a more specific

relation, most probably in terms of a dimensionless number similar to Froude number.

5- Additional laboratory experiments and field works are required to examine the sediment transporting energy head and to determine a functional relation between the sediment type and the transporting head. Furthermore, the dimensionless critical shear stress parameter can be related for the different types of sediment.

6- Other sediment transport predictors of access shear stress type can be incorporated in the model using the transporting energy concept instead of the friction energy.

7- More investigations are required for the treatment of the active layer and accordingly to study the armoring phenomena in alluvial channels.

8- A similar conceptual developments can be searched considering the suspended sediment in alluvial channels accounting for the computation of the bed load sediment beside.

9- The formulated model can be reconsidered using any suitable implicit scheme for the numerical solution of the model for coupled hydrodynamic and sediment phases.

10-More investigation and analysis should be carried on alluvial channels with smaller bed size.

11-The newly developed concepts can be extended for three-dimensional models starting from the basic principles.

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APPENDIX (A)

SEDTREN MODEL SUBROUTINES

This appendix introduces brief description for each subroutine used in the computer code of SEDTREN model.

FLOPARM

This is a subroutine to compute the flow parameters, the flow velocity, water level and the energy head loss in each section along the channel.

TRENHED

This subroutine computes the transporting energy head in each section along the channel reach.

REENHED

This subroutine determines the resisting energy head in each section along the channel reach.

ENESLOP

This is a subroutine to evaluate the slopes of the transporting energy and resisting energy lines at each section along the channel.

FROREYN

This is a subroutine to compute the flow characteristics, Froude Number, Reynolds Number and Reynolds Sediment Particles Number.

MIXLAYR

This is a subroutine to determine the thickness of the mixing layer and sediment particles distribution in the layer.

SEDRATE

This subroutine computes the rate of the sediment transport in each section along the channel reach.

AGGRDEG

This is a subroutine to determine the erosion and the deposition thickness in each sub-reach along the channel.

BEDCHNG

This subroutine computes the change in bed level occurs in each sub-reach along the channel.

NEWTPRO

This subroutine evaluates the new water surface profile along the channel reach.

APPENDIX (B)

BED-LOAD TRANSPORT

For bed-load transport, the basic modes of particles motion are sliding rolling or saltation motion. The sediment transport rate may be measured by weight, mass or volume. In practice, it is often expressed per unit width of the channel either by mass or by volume. These are related by:

$$m_s = \rho_s . q_s \qquad \text{.........(B.1)}$$

Where; m_s is the mass of sediment flow per unit width, ρ_s is density of sediment particles and q_s is the volumetric sediment discharge per unit width. Bed-load transport occurs when the bed shear stress, τ_O, exceeds the critical shear stress, $(\tau_O)_c$. In a dimensionless form the shear stress is expressed as follow:

$$\tau^* = \tau / (\gamma_s (ss - 1) d_s) \qquad \text{.........(B.2)}$$

This dimensionless shear stress should exceed the dimensionless shear stress parameter, τ_c^*, for the initiation of bed-load transport.

Bed-load transport rate is associated with inter-granular forces and takes place in the active layer. During the motion, the moving particles are subject to hydrodynamic forces, gravity force and inter-granular forces. Chanson (1999) schematized clearly the bed-load motion, as shown in figure (B.1), and expressed that, the normal stress exerted by the bed-load on the immobile bed particles is called the effective stress, σ_e, and proportional to

$$\sigma_e \propto \gamma\gamma_s (ss-1) C_s \cos\theta \qquad \text{..(B.3)}$$

Where; C_s is the volumetric concentration of the sediment in the bed-load layer and $\cos\theta$ is the longitudinal bed slope. Physically, the transport rate is related to the characteristics of the bed-load layer, the thickness of the layer, δ_s, and the average speed of the sediment moving along the bed.

The normal stress increases the frictional strength of the sediment bed and the boundary shear stress

applied to the top of the immobile particles layer becomes:

$$\tau_o = (\tau_o)_c + \sigma_e \tan\phi_s \quad \text{...(B.4)}$$

Where, $\tan\phi_s$ is the angle of repose of sediment particles.

Several researchers proposed formulae to estimate the characteristics of the bed-load layer. Table (B.1), presented by Chanson (1999), shows a comparison between some formulae concerning the bed-load layer characteristics. On the other hand, attempts of several researchers to predict the bed-load transport rate were shown also by Chanson (1999) in table (B.2).

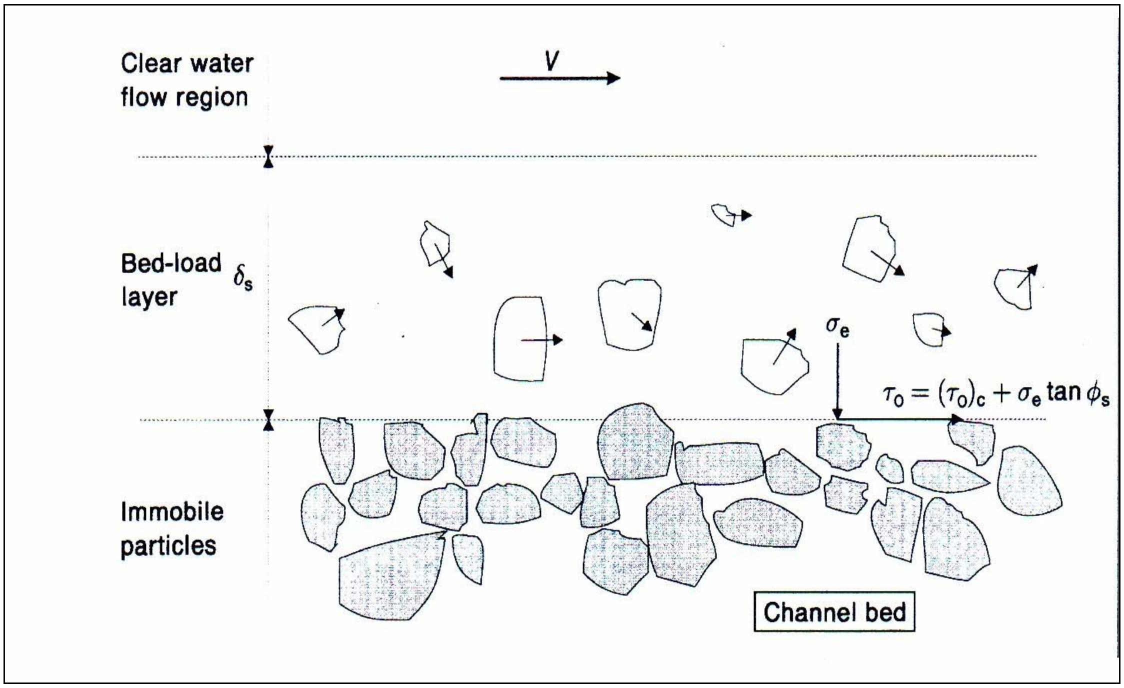

Fig. (B.1) Motion Of Bed-Load Sediment

{Source: Chanson (1999).}

Table (B.1) Bed-Load Layer Characteristics

Reference (1)	Bed-load layer characteristics (2)	Remarks (3)
Fernandez-Luque and van Beek (1976)	$\frac{V_s}{V_*} = 9.2\left(1 - 0.7\sqrt{\frac{(\tau_*)_c}{\tau_*}}\right)$	Laboratory data $1.34 \leq \mathbf{s} \leq 4.58$ $0.9 \leq d_s \leq 3.3\,\mathrm{mm}$ $0.08 \leq d \leq 0.12\,\mathrm{m}$
Nielsen (1992)	$C_s = 0.65$ $\frac{\delta_s}{d_s} = 2.5(\tau_* - (\tau_*)_c)$ $\frac{V_s}{V_*} = 4.8$	Simplified model.
Van Rijn (1984a,1993)	$C_s = \frac{0.117}{d_s}\left(\frac{\nu^2}{(\mathbf{s}-1)g}\right)^{1/3}\left(\frac{\tau_*}{(\tau_*)_c} - 1\right)$ $\frac{\delta_s}{d_s} = 0.3\left(d_s\left(\frac{(\mathbf{s}-1)g}{\nu^2}\right)^{1/3}\right)^{0.7}\sqrt{\frac{\tau_*}{(\tau_*)_c} - 1}$ $\frac{V_s}{V_*} = 9 + 2.6\log_{10}\left(d_s\left(\frac{(\mathbf{s}-1)g}{\nu^2}\right)^{1/3}\right) - 8\sqrt{\frac{(\tau_*)_c}{\tau_*}}$	For $\frac{\tau_*}{(\tau_*)_c} < 2$ and $d_s = d_{50}$. Based on laboratory data $0.2 \leq d_s \leq 2\,\mathrm{mm}$ $d > 0.1\,\mathrm{m}$ $Fr < 0.9$
	$C_s = \frac{0.117}{d_s}\left(\frac{\nu^2}{(\mathbf{s}-1)g}\right)^{1/3}\left(\frac{\tau_*}{(\tau_*)_c} - 1\right)$ $\frac{\delta_s}{d_s} = 0.3\left(d_s\left(\frac{(s-1)g}{\nu^2}\right)^{1/3}\right)^{0.7}\sqrt{\frac{\tau_*}{(\tau_*)_c} - 1}$ $\frac{V_s}{V_*} = 7$	$d_s = d_{50}$. Based on laboratory data $0.2 \leq d_s \leq 2\,\mathrm{mm}$ $d > 0.1\,\mathrm{m}$ $Fr < 0.9$

Notes: V_* = shear velocity; $(\tau_*)_c$ = critical Shields parameter for initiation of bed load.

{Source: Chanson (1999).}

Table (B.2) Bed-Load Transport Formulae

Reference (1)	Formulation (2)	Range (3)	Remarks (4)
Boys (1879)	$q_s = \lambda\tau_o(\tau_o - (\tau_o)_c)$		λ was called the characteristic sediment coefficient.
	$\lambda = \frac{0.54}{(\rho_s - \rho)g}$ Schoklitsch (1914)		Laboratory experiments with uniform grains of various kinds of sand and porcelain.
	$\lambda \propto d_s^{-3/4}$ Straub (1935)	$0.125 < d_s < 4\,\text{mm}$	Based upon laboratory data.
Schoklitsch (1930)	$q_s = \lambda'(\sin\theta)^k(q - q_c)$ $q_c = 1.944 \times 10^{-2} d_s(\sin\theta)^{-4/3}$	$0.305 < d_s < 7.02\,\text{mm}$	Based upon laboratory experiments.
Shields (1936)	$\frac{q_s}{q} = 10\frac{\sin\theta}{s}\frac{\tau_o - (\tau_o)_c}{\rho g(\mathbf{s}-1)d_s}$	$1.06 < \mathbf{s} < 4.25$ $1.56 < d_s < 2.47\,\text{mm}$	
Einstein (1942)	$\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} = 2.15\exp\left(-0.391\frac{\rho(\mathbf{s}-1)gd_s}{\tau_o}\right)$	$\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} < 0.4$ $1.25 < \mathbf{s} < 4.25$ $0.315 < d_s < 28.6\,\text{mm}$	Laboratory experiments. Weak sediment transport formula for sand mixtures. Note: $d_s \approx d_{35}$ to d_{45}.
Meyer-Peter (1949,1951)	$\frac{\dot{m}^{2/3}\sin\theta}{d_s} - 9.57(\rho g(\mathbf{s}-1))^{10/9} = 0.462(\mathbf{s}-1)\frac{(\rho g(\dot{m}_s)^2)^{2/3}}{d_s}$	$1.25 < \mathbf{s} < 4.2$	Laboratory experiments. Uniform grain size distribution.
	$\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} = \left(\frac{4\tau_o}{\rho(\mathbf{s}-1)gd_s} - 0.188\right)^{3/2}$		Laboratory experiments. Particle mixtures. Note: $d_s \approx d_{50}$.
Einstein (1950)	Design chart $\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} = f\left(\frac{\rho(\mathbf{s}-1)gd_s}{\tau_o}\right)$	$\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} < 10$ $1.25 < \mathbf{s} < 4.25$ $0.315 < d_s < 28.6\,\text{mm}$	Laboratory experiments. For sand mixtures. Note: $d_s \approx d_{35}$ to d_{45}.
Schoklitsch (1950)	$\dot{m}_s = 2500(\sin\theta)^{3/2}(q - q_c)$ $q_c = 0.26(\mathbf{s}-1)^{5/3}d_{40}^{3/2}(\sin\theta)^{-7/6}$		Based upon laboratory experiments and field measurements (Danube and Aare rivers).
Nielsen (1992)	$\frac{q_s}{\sqrt{(\mathbf{s}-1)gd_s^3}} = \left(\frac{12\tau_o}{\rho(\mathbf{s}-1)gd_s} - 0.05\right)\sqrt{\frac{\tau_o}{\rho(\mathbf{s}-1)gd_s}}$	$1.25 < \mathbf{s} < 4.22$ $0.69 < d_s < 28.7\,\text{mm}$	Re-analysis of laboratory data.

Notes: $\dot{m}$ = mass water flow rate per unit width; $\dot{m}_s$ = mass sediment flow rate per unit width; q = volumetric water discharge; q_s = volumetric sediment discharge per unit width; $(\tau_o)_c$ = critical bed shear stress for initiation of bed load.

{Source: Chanson (1999).}